PROCEEDINGS
THE INSTITUTION OF CIVIL ENGINEERS

PART I
JULY 1955

ORDINARY MEETING

15 February, 1955

DAVID MOWAT WATSON, B.Sc., President, in the Chair

The Council reported that they had recently transferred to the class of

Members

BOWDLER, HENRY.
BUCK, ALFRED CECIL, M.Sc.(Eng.) (*London*).
EVANS, GEORGE RICHARD.
GUILD, ALEXANDER EDDIE, B.Sc. (*Aberdeen*).
HENRY, JOHN KEITH MAXWELL, B.A.I. (*Dublin*).
MOFFATT, HENRY ROBERT, M.Sc. (*Leeds*).
OWEN, ARTHUR LLOYD.
PITT, ROBERT BRINDLEY, C.B.E., M.C., T.D., J.P., D.L., B.A. (*Cantab.*).
WHITTINGHAM, WILLIAM STANLEY, M.Eng. (*Liverpool*).

and had admitted as

Graduates

AITKEN, IAN BRYCE, B.Sc. (*Glasgow*), Stud.I.C.E.
ARULIAH, PANCHANAYAGAM, B.Sc. (*Ceylon*), Stud.I.C.E.
ATKINSON, CHARLES HARRISON, B.Sc. (*Durham*), Stud.I.C.E.
BAILEY, BRIAN CHARLES TAYLOR, Stud. I.C.E.
BAILEY, DEREK BRUCE, B.Sc.(Eng.) (*London*), Stud.I.C.E.
BAILEY, JOHN CLIFFORD, B.Sc.(Eng.) (*London*).
BANHAM, DAVID RICHARD, Stud.I.C.E.
BARKER, MICHAEL, B.Sc. (*Nottingham*).
BARTLETT, CHRISTOPHER HENRY, B.A. (*Cantab.*).
BENNETT, CLIVE, B.Sc.(Eng.) (*London*).
BISH, GERALD MALCOLM, B.Sc.Tech. (*Manchester*).
BLACK, JOHN MASON, B.Eng. (*Liverpool*).
BLACKMORE, NORMAN ROBERT.
BOWDEN, JAMES FARQUHAR, Stud.I.C.E.
BUILT, JOHN WILLIAM, B.Sc.(Eng.) (*London*).
BURCH, PHILIP EDWARD, B.Sc. (*Bristol*).
BURROWS, FREDERICK GEORGE, B.Sc. (Eng.) (*London*), Stud.I.C.E.
CAMERON, GORDON ROSS, B.Sc.(Eng.) (*London*), Stud.I.C.E.
CAMPKIN, WILLIAM DENIS, B.Sc.(Eng.) (*London*), Stud.I.C.E.
CHALCRAFT, WILLIAM HEWSHOTT, B.Sc. (Eng.) (*London*).
CLOUGH, IAN RAYSON, B.Eng. (*Liverpool*).

Croad, Ernest Norman.
Day, Graham, B.Sc. (*Nottingham*).
De Silva, Douglas Sepala, Stud.I.C.E.
Dilks, John Roxby, B.Sc. (*Durham*).
Dixon, Victor Donald, Stud.I.C.E.
Donaldson, Peter Kenyon, Stud.I.C.E.
Dyer, Peter Frederick, Stud.I.C.E.
Edwards, Peter Scriven, B.Sc. (*Manchester*).
Elsdale, Henry Robinson, B.A. (*Cantab.*).
Emerson, Francis William, Stud.I.C.E.
Evans, John Hopwood, B.Sc. (*Birmingham*).
Finch, Alan, B.Sc.(Eng.) (*London*).
Fisher, Roger, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Fitzsimon, Leo Andrew, B.E. (*National*).
Ford, Gerald Duncan, Stud.I.C.E.
Fox, Noel Howard, Stud.I.C.E.
Greening, Derek Miles, Stud.I.C.E.
Hall, John Douglas, B.Sc. (*Durham*), Stud.I.C.E.
Hamilton, John McCulloch, B.Sc. (*Glasgow*), Stud.I.C.E.
Harmon, Philip Joseph, B.E. (*National*).
Harper, Victor Francis George.
Harris, John, B.Sc. (*Bristol*).
Harrison, James Geoffrey, B.Sc. (*Leeds*), Stud.I.C.E.
Hayward, Gordon Douglas, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Hillier, Donald Arthur, Stud.I.C.E.
Hoggan, James Imrie, Stud.I.C.E.
Holland, Jack, B.Sc. (*Leeds*), Stud.I.C.E.
Hook, David Morgan Alfred, B.A. (*Cantab.*), Stud.I.C.E.
Hyde, Geoffrey Hammond, B.E. (*New Zealand*).
Hyde, John MacLeod, Stud.I.C.E.
James, Alexander Souter, B.Sc. (*Aberdeen*).
James, Derek Charles Roy, B.Sc.(Eng.) (*London*), Stud.I.C.E.
James, Thomas Michael Dennis, B.Sc. (*Nottingham*).
Johnson, Harold Graham, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Jones, Colin Edgar, Stud.I.C.E.
Keery, George Shaw, B.Sc. (*Belfast*), Stud.I.C.E.
Kierulf, Haakon, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Lancaster, Robert Mortimer, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Langford, James Edward, B.Eng. (*Liverpool*), Stud.I.C.E.
Law, John Dickson, B.Sc. (*Aberdeen*).
Low, Ernest, B.Sc. (*Hong Kong*).
McCall, John McLachlan, B.Sc. (*Edinburgh*), Stud.I.C.E.
Mackay, Matthew, B.Sc. (*Aberdeen*).
McLeman, William John Murdoch, B.Sc. (*Aberdeen*).
McMurdo, John, B.Sc. (*Glasgow*).
Mardon, John Gale, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Mayhew, Donald Wesley, Stud.I.C.E.
Meacock, George Johnson Cann, B.A., B.A.I. (*Dublin*).
Middle, John Nelsey, Stud.I.C.E.
Mitchell, Alan Ogston, B.Sc. (*Aberdeen*).
Mitchell, Robert Hartley Ogilvie, B.Sc. (*Aberdeen*), Stud.I.C.E.
Murphy, Diarmid Aidan Thomas, B.Eng. (*Liverpool*).
Nequest, Peter, Stud.I.C.E.
Nicoll, Murray, B.Sc. (*Edinburgh*), Stud.I.C.E.
North, Bernard Harry, Stud.I.C.E.
Nutland, John Henry Douglas, Stud.I.C.E.
O'Driscoll, Edward, B.E. (*National*).
Ogilvy, John Gow, B.Sc. (*St Andrews*), Stud.I.C.E.
Oliver, Roger Henderson, Stud.I.C.E.
Papadopoulos, John Aristocles, B.Sc. (*Glasgow*).
Parker, Arthur Vernon, Stud.I.C.E.
Parkes, David Peers, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Parkinson, Joseph Maurice, B.Sc. (*Durham*), Stud.I.C.E.
Phillips, David Watson, Stud.I.C.E.
Polack, Bernard McCarthy, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Roberts, Lynn, Stud.I.C.E.
Robertson, Ian, Stud.I.C.E.
Rooney, James Desmond, B.Sc.(Eng.) (*London*).
Rothwell, Miles Alan DeWeldycz, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Rowlands, Gruffydd Owen, B.Sc. (*Wales*).
Sanders, Richard Watkinson, B.Sc. (*Bristol*).
Savage, James Kerr Hamilton.
Scribbans, Reginald Glyn, Stud.I.C.E.
Simpson, Alan, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Smith, Ashley William Harris, Stud.I.C.E.
Smith, Denis Edward Thomas, B.Sc. (*Leeds*), Stud.I.C.E.
Smith, Gordon Raymond, B.Eng. (*Liverpool*).
Smith, James Gunning Eady, B.Sc. (*Belfast*).
Storey, John Douglas, Stud.I.C.E.

Strafford, Peter Thomas, Stud.I.C.E.
Taylor, James Gordon, B.Sc.(Eng.) (*London*).
Taylor, Ronald John, Stud.I.C.E.
Thompson, John Neville, B.Sc.(Eng.) (*London*).
Tyler, Maurice Anthony, B.Sc.(Eng.) (*London*).
Tyson, James Easten, Stud.I.C.E.
Waide, Keith Nelson, B.Sc. (*Durham*).
Whiteside, William Jamie.
Wilce, Brian John, B.Sc. (*Birmingham*).
Wilson, Derek, Stud.I.C.E.
Winters, Trevor, Stud.I.C.E.
Yip Shing-Pui, B.Sc. (*Hong Kong*).
Young, William Alexander, B.Sc. (*Glasgow*), Stud.I.C.E.

and had admitted as

Students

Bald, James Anderson.
Boardman, Michael Francis.
Bower, Edwin Charles.
Branch, Victor Oswald.
Bull, Kenneth John.
Campbell, John Young.
Carlin, James.
Caton, James Morphet.
Charlton, John Ashfield.
Coates, David Bernard.
Collings, Brian David.
Coster, Peter John.
Cowgill, Kenneth.
Davenport, Peter Ewart.
Douglas, John Glynn.
Dunlop, John Irvine.
Evans, John Seery.
Farrimond, George.
Gregory, Peter William.
Handley, John.
Hattersley, David Roberts.
Hill, John Collis.
Hughes, Brian Walter.
Hume, Peter Thomas.
Jenkinson, Geoffrey Eric.
Jones, George William.
Jones, Mervyn.
Just, Jack Tama Haigh.
Kafaru, Mudashiru Kokumo.
Keld, Terence Norman.
Kenney, David.
Kiln, John.
Kilpatrick, Hugh Michael.
Kite, Derek Peter.
Lashmar, Peter Donald Lush.
Lazenbury, John Gordon.
Maber, Charles Arthur.
McClure, Alan John Campbell.
McConnell-Wood, George Alexander.
McEntee, John Michael.
Maddox, Peter.
Madedor, Anthony Omunwarure.
Mesida, Samuel Olusanya.
Metcalf, John Beaumont.
Moore, Richard Arthur.
Morison, Robert Neil.
Morreau, Patrick Mark.
Moses, Gordon William.
O'Dwyer, Edward Francis.
Ojukwu, Emmanuel Nwokoye.
Oldfield, Peter John.
Oliver, Geoffrey Bryan Morgan.
Pai, Gurpur Ramachandra, B.E. (*Madras*).
Pawsey, Derek Brian Harold.
Phipps, Michael Thurston.
Pilling, James Frederick.
Plumb, David John.
Pocock, Humphrey Philip.
Postlethwaite, Roger William.
Rees, David Colin.
Rich, Michael Mervyn.
Roberts, Raymond David.
Saunders, Charles Robert.
Shadwell, Norman Frank Vernon.
Sharp, Keith Drummond.
Shaw, Neville.
Sidey, Alastair McIntosh.
Sims, John Alan.
Smith, Colin Brownfield.
South, Ronald James.
Starzewski, Krzysztof.
Stirling, John McClumpha.
Taylor, David James.
Taylor, Malcolm Frederick.
Taylor, Malcolm John.
Taylor, Roger Frank.
Thompson, Alan.
Veevers, Richard Harrison.
Walls, Benjamin John Stuart.
Whamond, Duncan Alan Alisdair.
Woodward, John Colin.
Wright, Jeremy Martin.
Young, Maurice Edwin.

The following Paper was presented for discussion and, on the motion of the President, the thanks of the Institution were accorded to the Authors.

Paper No. 6027

INVESTIGATIONS INTO THE DESIGN OF PRESSURE TUNNELS IN LONDON CLAY

by

***Fred Tattersall, M.Sc., M.I.C.E.,**
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SYNOPSIS

The Paper describes an experimental investigation into the behaviour of a hydraulic pressure tunnel which was designed and constructed by the Metropolitan Water Board at Ashford Common, Middlesex. The tunnel is 90 ft deep in the London Clay and is lined only with a precast concrete lining 8-ft-4-in. internal dia. which is incapable of carrying any hoop tension. The pressure of the water is therefore transmitted directly to the clay. The object of the investigation was to determine whether an inner steel lining was necessary, and generally to increase existing knowledge of the earth pressures and internal forces on such tunnel linings.

In collaboration with the Building Research Station special instruments were designed and constructed to measure directly radial earth pressures, hoop thrust, and diameter changes in the lining, both with the tunnel empty and under pressure. Most of the instruments were remote reading at the surface. The tunnel has been subjected to a head of 160 ft of water.

The mechanics of expanding and prestressing the lining have been investigated and suggestions are made for improving this type of tunnel lining. The Paper concludes with a discussion of the use and economics of tunnels of this type for the conveyance of water.

INTRODUCTION

THE use of tunnels for conveying water for public supply in the London area has not until recently been much favoured. There were many reasons for this. Tunnels were generally considered to be far more expensive than surface pipes and it is necessary to make them of a certain minimum diameter, which is large compared with most surface conduits. It was also usually more convenient for reasons of distribution to have a larger number of smaller conduits than a few large ones. Finally, most engineers probably considered the geological conditions not specially suitable for hydraulic pressure tunnels.

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During the 1939–45 war ideas began to develop amongst the engineering staff of the Metropolitan Water Board that tunnels would have to be used to an increasing extent in the future, particularly in the development of a scheme for conveying water from the River Thames to the Lee Valley. In this case a tunnel would be particularly advantageous because the discharge level is very little higher than the inlet level. The water could flow by gravity through the tunnel and need only be pumped at the discharge shaft. Thus, the pressure in the tunnel would be low and, since the intervening ground along the route of the tunnel is mostly higher than at the two ends, ample cover would be available to retain the water in the tunnel.

The failure of the Sydney pressure tunnel,[1, 2] probably had an unwarranted deterrent effect on the development of pressure tunnels in relatively soft material such as London Clay. That tunnel failed as a result of a combination of unsound rock and a lining that had cavities behind it, and which would take little or no tension. London Clay, on the other hand, is homogeneous strong and watertight and, depending on the depth of the tunnel, was expected to exert a pressure on the lining sufficient to balance the water pressure. Alternatively, it seemed possible to induce a compression both in the clay and the lining in the course of erection. The lining would thus become prestressed and, providing this was sufficient, would remain in compression and undisturbed under internal water pressure. Thus, the need for a tensile lining could be avoided, provided that the clay was sufficiently watertight and protected from erosion. Since a steel lining nearly doubles the cost of a tunnel of this type, for a given diameter, the importance of this will be apparent.

A short length of tunnel lined with precast concrete segments of the type used in the Ashford Common Tunnel had previously been driven at Stoke Newington. This tunnel was lined with steel pipes, most of them being ½ in. thick, the space between the pipe and the concrete segments being filled with colloidal cement grout. The work demonstrated the practicability of this type of lining, but provided no information about the stresses in either the linings or their behaviour, it being assumed at this time that a steel lining was necessary. This work has been described recently by Scott.[3]

Ashford Common Tunnel

The Ashford Common Tunnel (see Fig. 1), on which most of the work described in this Paper was carried out, is intended to form the main supply from Queen Mary reservoir to the new Ashford Common filtration works and is 1,882·6 ft long between centres of shafts. The ground level along the line of the tunnel is about 41 O.D., and the normal hydraulic-gradient line in service will be between 70 and 74 O.D. It was designed to carry

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[1] The references are given on p. 455.

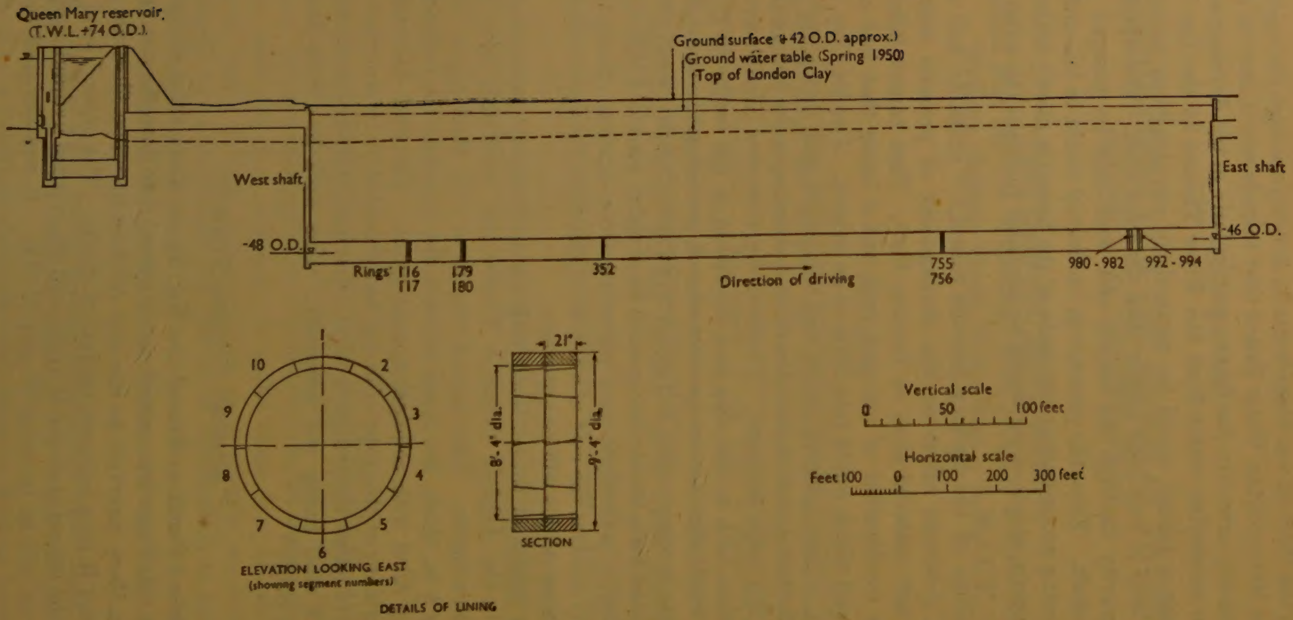


FIG. 1.—LONGITUDINAL SECTION OF THE TUNNEL AND DETAILS OF LINING

TABLE 1.—POSITIONS OF GAUGES (see Fig. 1)

Ring	Pressure measurement gauges			No. of diameter gauges	No. of joint-movement gauges	
	Type	No.	Positions: segments		Wedge type	Link type
116	Radial hydraulic .	5	1, 3, 5, 7, 9	—	2	—
117	Radial hydraulic .	5	2, 4, 6, 8, 10	—	2	—
179	Radial vibrating wire.	5	1, 3, 5, 7, 9	3	2	5
180	Radial vibrating wire.	5	2, 4, 6, 8, 10	3	2	4
352	Radial vibrating wire.	10	Every segment	5	3	10
755	Hoop-thrust vibrating wire . . .	2 pairs	4, 6	3	3	5
	Radial hydraulic .	5	1, 3, 5, 7, 9			
756	Hoop-thrust vibrating wire . . .	2 pairs	6, 8	3	3	5
	Radial hydraulic .	5	2, 4, 5, 7, 10			
790	—	—	—	—	4	—
981	Hoop-thrust vibrating wire . . .	1 pair	4	3	2	—
993	Hoop-thrust vibrating wire . . .	1 pair	4	—	2	—

Notes :—1. Two thermistors were installed at each of rings 352 and 981.
2. Two 2-in.-dia. sampling holes were cast in segment 3 of ring 981 and rings 993 to 1,021 inclusive.

TABLE 2.—POSITIONS OF EXPERIMENTAL CEMENT INNER LININGS (see Fig. 1)

Position	Length: ft	Thickness: in.	Description
From west shaft . .	50	1	Unreinforced gunite.
From east shaft . .	50	2	Gunite reinforced by a steel mesh.
Near ring 700 . .	4	¾	Cement rendering applied by hand.
Near ring 700. . .	4	¾	Cement rendering applied by hand over a priming coat of latex cement.

200 m.g.d. and, to deal with possible future hydraulic conditions, it would in any case have been necessary to construct even a surface conduit at a considerable depth. It was, therefore, decided to construct a deep tunnel and to take advantage of the earth pressure to resist internal pressure. Construction of the tunnel began in January 1952 and was completed in May 1952. The tunnel was first filled in March 1953.

The tunnel lining was of the Don-Seg type described by Scott.[3] Each ring had an internal diameter of 100 in., an external diameter of 112 in., a length of 21 in., and consisted of ten equal wedge-shaped segments. Each segment was 33·5 in. along the intrados at the wide end, 29·3 in. at the narrow end, 21 in. longitudinally, 6 in. thick, and weighed about 360 lb. The longitudinal edges were brushed with bitumen paint and allowed to dry before erection. Alternate segments were placed in a ring with the wide end against the last assembled ring and the ring was then tightened by using the shield jacks to wedge home the intermediate segments with their narrow ends towards the last assembled ring. No special order of closing was specified and different gangers used different methods.

Experience at Stoke Newington had shown that it was difficult to erect a lining of this type with tight joints and at Ashford Common it was originally intended to line the segments internally with 2 in. of gunite, making the finished diameter 96 in. This inner lining could not be placed until all instruments had been removed and, as an interim measure, all open joints were pointed with mortar as well as possible, a final decision on the internal surfacing of the tunnel being delayed to a later date.

Borings put down on the line of the tunnel proved the existence of a surface layer of water-bearing gravel 15 to 20 ft thick and, below that, hard London Clay to below the line of the tunnel. Other geological records for the area indicate that the London Clay extends to about 350 ft below the surface.

Basis of design

Very little data existed on the earth pressures to be expected on tunnel linings in London Clay at such a depth. It was generally thought that, after a sufficient lapse of time, the pressure on the tunnel lining would approximate to a pressure equivalent to the full depth of overburden. The basic assumption of design was that the earth pressure on the tunnel rings should be not less than the water pressure. On the assumption that the internal surfacing of the lining was watertight the tunnel rings would not then be subjected to tension since the water pressure would merely reduce the compressive stress arising from earth pressure. There are, of course, no connexions such as bolts, between the segments, and the tunnel rings are incapable of taking any tension.

If the lining or its internal surfacing were not watertight, it should make no appreciable difference to the stresses in the tunnel segments. The only difference would be that the water pressure would then tend to act directly against the clay instead of acting through the segments, but would similarly reduce the compressive hoop stress on the lining.

The normal water head on the tunnel will be 74 O.D., with ground level approximately 41 O.D. Assuming the weight of earth is double that of water, and that ultimately the earth pressure would equal overburden pressure, the water pressure would equal the earth pressure with the tunnel at a depth of 33 ft. There was, however, a possibility that the head could rise to about 100 O.D. and this pressure would require a depth of 59 ft.

The cost of placing the tunnel at a greater depth was comparatively small and it was decided to drive it with a level axis at about 90 ft below ground level. This was considered to give an adequate factor of safety. It was decided also that the London Clay could safely be relied upon for watertightness. The clay is fissured but the fissures are closed in the undisturbed condition and it was considered that no significant leakage could take place through the 70 ft of clay overlying the tunnel.

The tunnel was driven wholly from the west shaft. The shafts had an internal diameter of 12 ft and were lined with bolted precast concrete segments. Subsequently, when the tunnel had been driven, a thin welded steel pipe of 9-ft dia was placed inside the shafts and turned about 12 ft inside the tunnel, the space between the steel pipe and shaft lining being filled with concrete. Four short lengths of cement linings of different types were applied in the tunnel; details of these are given in Table 2.

Shield

The tunnel was shield-driven, the shield had ten hydraulic rams, one to each segment in the ring. The usual function of a shield is to act as a protection against bad ground, but in this type of tunnel where the lining is precast and is not grouted, the primary function of the shield is to cut a circular hole of an exact diameter. The protective function is secondary, because this type of lining would not be used in bad ground. This function of the shield has not hitherto been given sufficient consideration. In the Authors' view the methods normally used for fabricating shields are not always sufficiently accurate for a precast concrete lining of fixed diameter driven in hard clay.

When the clay is exposed behind the shield the pressure is released and the clay expands. When the lining is erected and tightened the clay is, to some extent, recompressed. It is important, therefore, to get the correct relation between shield diameter and lining diameter, which makes proper allowance for expansion of the clay and produces the required degree of compression in the ring and corresponding pressure against the clay.

Erection of lining

The method of closing a tunnel ring of this type involves considerable movement of the individual segments both circumferentially and longitudinally after the segments are first placed in position. This movement may be 5 in. longitudinally and 3 in. circumferentially, and takes place under varying loads. The friction between the materials is high and it will be shown that more than 80% of the final jacking force was lost in friction. The effect of these movements under load is that it is difficult to keep the segments in their correct final positions in the ring and bearing uniformly against each other.

The final positional errors may be small but this type of ring requires an exact diameter and, unless this is achieved, the circumference of the ring becomes irregularly larger and difficult to close. The longitudinal joints become eccentrically loaded with high local stresses and the risk of spalling increases. If a segment is not driven fully home, the protrusion of the segment into the transverse joint interferes with the correct positioning of succeeding rings and the transverse gaps tend to increase. As a result of these factors the final jacking pressure fluctuates considerably from ring to ring, the average final jacking pressure used ranging from about 2,000 to 2,500 lb/sq. in. on a 7-in.-dia. ram. Sometimes the pressure went up to the limit of the gauge, 3,500 lb/sq. in., and occasionally even this did not suffice to close the ring completely.

Transverse gaps between segments also arise from corrections of line and level. Such gaps may be the source of erosion behind the linings in water tunnels. The Authors suggest that corrections of line and level need be made much less frequently in water tunnels than in railway tunnels. Considerable errors are of no great consequence, provided that the invert will drain satisfactorily when the tunnel is empty, whilst a reduction in the number of corrections would enable better fitting of rings to be obtained.

Experimental investigations

The full scope of the experimental work that would be necessary could not be envisaged at the outset and much additional work had to be undertaken. Modifications were necessary and new instruments were designed and made as construction proceeded and more information was gained, but the experimental work could not be allowed to interfere with tunnelling which proceeded continuously.

Shortage of time eventually limited what could be attempted and, in some cases, there was insufficient time for the complete development and trial of new instruments. Comparatively little damage was done to the instruments and both contractors and men were very co-operative in all the experimental work.

The prime object of the investigation was to find out whether it was necessary to line such a tunnel with a steel waterproof inner lining to resist internal water pressure. Secondly, it was desired to obtain as much information as possible on the behaviour of such tunnel linings when used both in dry tunnels and hydraulic tunnels under pressure.

The broad outline of the investigation was as follows :—

(1) To measure the direct radial earth pressure on the lining and record its change with time.

(2) To measure the hoop thrust in the tunnel lining and similarly to record its change with time.

(3) To measure the deformation of the rings with the tunnel empty.

(4) To investigate the mechanics of erecting the lining.

(5) To investigate the changes in earth pressure, hoop thrust, and movement of the lining as the tunnel is charged with water.

LABORATORY TESTS ON THE CLAY

Borings made before the tunnel was constructed showed that the London Clay was rather variable in character, being more silty in some places than others. The weathered brown clay under the flood-plain gravel was only about 6 in. thick.

A plot of the water-content distribution from three borings showed no consistent variation with depth. It ranged from 23 to 27% of dry weight at about 100 ft below surface, and even just below the gravel the water content was not outside this range. The greatest variation (21 to 28%) was at a depth of about 50 ft.

The density variation followed inversely the water-content variation; the greatest range (123–129 lb/cu. ft) was at about 50 ft and at 100 ft it varied from 127 to 129 lb/cu. ft.

Undrained triaxial compression tests made on pairs of 1½-in.-dia. samples extracted from 4-in.-dia. cores taken from 80 to 100 ft depth were generally less than one-third the strength of samples obtained from the tunnel and trimmed by hand. Even at a depth of only 40 ft at this site the tests on cored samples probably underestimated the strength considerably. The material was too hard for core samples to be undisturbed.

The following features were noted in the clay as it was extracted at the tunnel face.

A few claystones (calcite) were encountered, and in particular one fairly extensive band in the vicinity of ring 352. In some parts there were numbers of shells and worm casts.

Close examination of the hard fissured clay revealed, in places, tiny silty pockets and layers, and the material generally was very faintly laminated in a horizontal plane.

Three extensive planes, glistening with moisture, were seen in the tunnel and there were probably more that were not seen. They were approximately parallel, extended completely across the face, and dipped westwards at about 80° to the tunnel axis. A thin film of softened clay, perhaps 1/16 in. thick and probably consolidated at the present overburden pressure, formed the boundaries of the plane. The planes were probably large-scale structural ruptures (faults) associated with slight tectonic movements and were quite different in character and origin to the fissures. The fissures had matt surfaces without any visible sign of a softened film.

Samples for mechanical testing were extracted as large intact lumps from the tunnel face, waxed, and taken to the laboratory. Here they were carefully sawn roughly to size and then gradually trimmed into solid cylinders in special trimming devices. The preparation of a sample took several hours. The orientation of the axes of the cylindrical samples with respect to the clay strata was at random.

Undrained triaxial compression tests were made on seventeen 1½-in. dia. and 3½-in. long samples of the clay, mostly at lateral pressures of 90 lb/sq. in. and at a strain rate of 1% in 72 sec. Initially, cycles of loading and unloading axial stress were applied up to about one-third the ultimate stress. After the first load application, the stress-strain curves were practically straight lines and a mean modulus of elasticity was obtained. Subsequently the samples were taken to failure.

The results of the tests on the clay taken from the tunnel are summarized in Table 3.

TABLE 3

	No. of samples	Mean	Range
Water content: % dry weight . .	17	21·6	14–25
Wet density: lb/cu. ft	17	130	127–135
Liquid limit: %	17	77	71–80
Plastic limit: %	17	29	27–32
Compression strength: lb/sq. in. . .	17	300	145–655
Strain at failure: %	17	2·7	1·9–4·5
Inclination of rupture plane to principal stress plane	15	59°	50–69°
Modulus of elasticity: lb/sq. in. . .	15	37,000	11,000–71,700

Curiously enough, the weakest samples failed at the smaller strains, which indicates that the lower strengths are associated with weak planes, probably fissures too fine to be noticed in some cases. Ruptures certainly occurred at fissure planes in some of the weaker samples.

Consolidation-swelling tests were made on several samples of the clay. The technique in general was to put the sample into the press and to load it up to values approximating to the critical pressure, i.e., to a pressure which would prevent either swelling or consolidation taking place when the water was admitted subsequently to the porous stones. If swelling occurred, the load was increased until consolidation occurred. The sample was then completely unloaded from a load above the critical pressure to give an equilibrium rebound curve.

It was found that loads of 4 to 7 tons/sq. ft led to consolidation and that loads of 3 to 6 tons/sq. ft caused the various samples to swell. The mean critical pressure was about 5·2 tons/sq. ft which corresponds closely with the total overburden pressure and which might occur under certain conditions of pore-water pressure distribution in the London Clay. The result suggests that the effective stress in the clay corresponds to the total overburden pressure, although there is some doubt because precautions taken to prevent very slight drying during sample preparation may not have been entirely successful.

The samples in these tests started at an average initial water content (% dry weight) of about 23·6 and varied between 23·2 and 24·1. Under the reduced effective stress of 2 tons/sq. ft, corresponding approximately to the loads subsequently measured between the tunnel lining and the clay, the equilibrium water content is about 24·0%. This increase of only about 0·4% in water content is difficult to detect in sampling when greater variations could be found at intervals of ½ in. or less in the virgin clay. The corresponding percentage of swelling is only 0·7% of volume.

INSTRUMENTS IN USE AND OBSERVATIONS DURING THE PERIOD WHEN THE TUNNEL WAS EMPTY

Instruments

The instruments installed in the tunnel were chosen and placed with the object of measuring the forces in the tunnel lining, the reactions between the clay and the lining, and the distortion of the lining.

The success of any experimental field observations in civil engineering work depends largely on the long-term stability and robustness of the instruments, and on the facility of placing them without disturbing the normal construction process. It is also necessary to install a sufficient number of instruments to cover the variations in load and displacement between different tunnel rings and even between one segment and the next.

The instruments had to meet a number of difficult conditions. In the first stage of the investigation when the tunnel was empty, air temperature changes, and consequently their effects on the instruments, were quite small. In the later stages, however, when the tunnel was filled with water, instrument conditions were much more severe. Observations had to be made from the ground surface while the instruments were subjected to high water pressures and varying temperatures.

It is desirable, when measuring the distortion of the tunnel lining, to measure the absolute movement in space of each segment but constructional operations prevented this, except for some special tests when individual tunnel rings were loaded circumferentially. Absolute measurements were impracticable when the tunnel was filled with water. Measurement of the distance between the five pairs of diametrically opposite segments is simple and this measurement was made throughout the investigation.

In addition to the above conditions, instruments for measuring the forces on the tunnel lining and within the lining should not disturb the actual forces by their presence. In other words the gauges need to be as stiff as the concrete of the segments, and the stiffer the clay the more important it is to meet this condition. The instruments need also to record with the minimum delay.

The best instrument known to the Authors, that meets the above conditions most closely, is the vibrating-wire load gauge, whose reliability had been demonstrated at Shellhaven.[4]

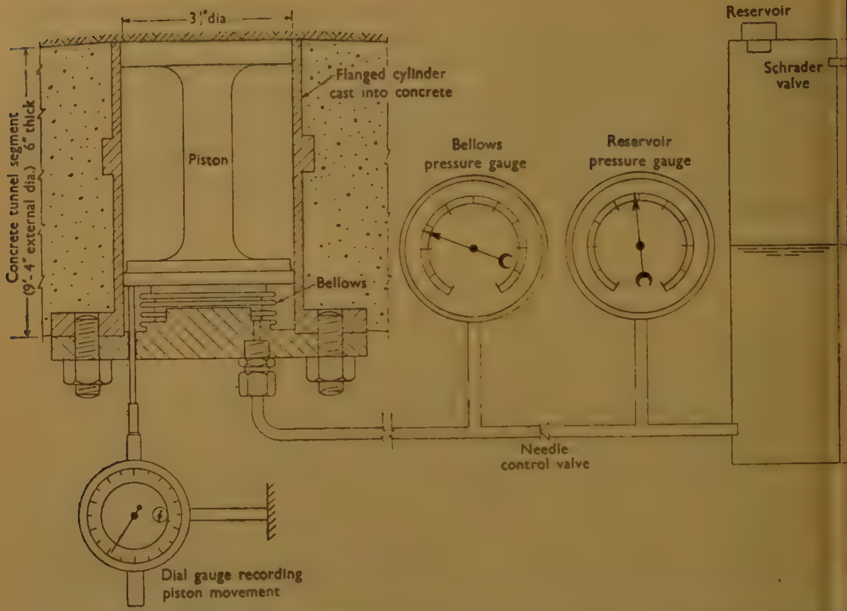

3½"dia
Reservoir
Schrader valve
Flanged cylinder cast into concrete
Bellows pressure gauge
Reservoir pressure gauge
Piston
Concrete tunnel segment (9'-4" external dia.) 6" thick
Bellows
Needle control valve
Dial gauge recording piston movement

FIG. 2.—HYDRAULIC RADIAL PRESSURE GAUGE AND DIAGRAM OF PRESSURE-RECORDING SYSTEM

However, since there was very little previous experience of measuring pressures on tunnel linings it was felt that a check on the vibrating-wire load gauges was desirable. For this reason one set of gauges using hydraulic load capsules was designed, although it was realized that such gauges could not be used when the tunnel was charged with water.

All instruments were constructed in the workshops of the Metropolitan Water Board and the Building Research Station.

Hydraulic radial pressure gauge

A section through the gauge mounted in a segment and a diagram of the external hydraulic connexions is shown in Fig. 2. It consists of a bronze cylinder of 3½-in. internal dia. and 6 in. long with a flange at one end, which is cast into the centre of a segment. A dumb-bell-shaped piston, with bearing surfaces at each end, slides accurately inside the cylinder, and is pushed against the clay by a three-ply seamless brass bellows at the inner end. The brass bellows which forms the hydraulic load capsule, is bolted to the flange of the bronze cylinder; it is connected by copper pipe to a Bourdon pressure gauge, and through a needle control valve to a water reservoir with another pressure gauge. The bellows and the hydraulic system as far as the reservoir are completely filled with air-free water.

In use the aim is to adjust the pressure in the bellows so that there is no relative movement between the piston and the segment from the time the segment is jacked into position. The piston was held with its outer end flush with the surface of the concrete segment by means of a spacing device in place of the bellows, until the ring was finally rammed into place. Subsequently the spacing device was removed, the hydraulic capsule fitted, and coupled by copper tube to the pressure gauge. This operation could last 12 hours and that time could elapse before the first trial pressure was applied to each hydraulic capsule. Air pressure was applied to the reservoir by means of a foot-pump and, by manipulation of the control valve and the reservoir pressure, the required pressure was applied to the bellows. One reservoir normally supplied a group of gauges, so the control valve to each gauge was shut after it had attained the desired pressure. Immediately the hydraulic capsule had been installed, a nominal pressure, equivalent to about 1·5 ton/sq. ft on the piston was applied and a 0·0001-in. dial gauge was set in position to record the piston movement. (See Fig. 2.) The bellows pressure was then adjusted at intervals in an attempt to prevent any piston movement.

The clay is so stiff in relation to the jacking forces applied to the segments that a good uniform contact between the clay and a segment cannot be expected and gaps were seen. Hence, to obtain a good representative value of the pressure between the clay and the segment the largest possible piston area is desirable. It was not possible, however, to use a larger piston because of the limited pressure capacity of commercial bellows.

Vibrating-wire radial pressure gauge

The load-sensitive unit of this gauge is a short mild-steel cylindrical tube, which carries the load axially. A thin wire is stretched along the axis of the cylinder and is anchored to circular plates fixed near each end of the cylinder. The wire is both plucked and its free vibration is picked up by the same small electromagnet (see Fig. 3). Axial compression of the steel tube causes a very small change in its length and hence in the frequency at which the stretched wire vibrates. The small movement required to operate this type of gauge is a very desirable feature.

The wire vibrations are amplified and fed on to a cathode-ray oscilloscope where they are balanced visually against the frequency of a reference gauge, consisting of a wire maintained in vibration. The frequency of the wire can be varied by spring loading from a screw micrometer. The load gauge is calibrated directly in a testing machine against readings of the screw micrometer. The calibration is almost linear and practically free from hysteresis.

It became almost standard practice to pluck the load gauge by the amplified output of the reference-gauge wire when at approximately the resonant frequency, instead of using a simple D.C. impulse. This method of exciting the wire is far more effective; a much larger amplitude is obtained on the load-gauge wire, and adjustment of the magnet clearance is not so critical.

The reference gauge proved insufficiently stable and it was necessary to check it at every reading against the nearest Lissajous figure obtained from a stable tuning fork. More recently, with the assistance of the Radio Research Station, a stable electronic oscillator was constructed from standard radio components. It promises to be more satisfactory and is certainly cheaper than a wire reference gauge.

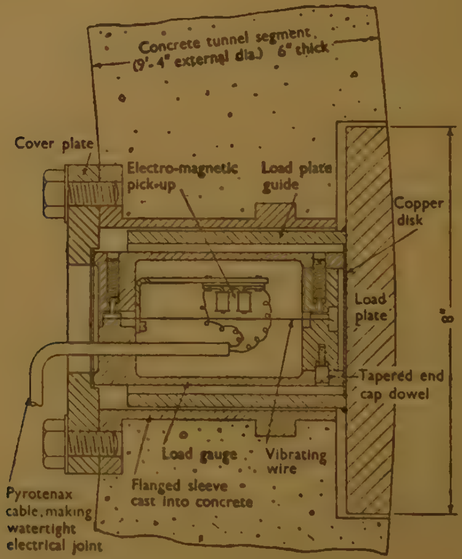

Concrete tunnel segment (9'-4" external dia.) 6" thick
Cover plate
Electro-magnetic pick-up
Load plate guide
Copper disk
Load plate
8"
Tapered end cap dowel
Load gauge
Vibrating wire
Flanged sleeve cast into concrete
Pyrotenax cable, making watertight electrical joint

Fig. 3.—Vibrating-wire radial pressure gauge

The load capacity of the gauge is set by selecting suitable dimensions for the cylinder wall thickness. The load sensitivity is inversely proportional to the length of the wire and, for any given length, is greatest when the wire is used throughout its lowest stable range of frequency. Fine wires are most stable at lower frequencies on account of their smaller flexural rigidity but, in other respects, the diameter of the wire has no effect on sensitivity.

The load gauges are self-compensating for changes in temperature. They were waterproofed, not entirely successfully, by soft soldering a thin cylinder of copper foil to thick copper bearing disks at each end of the load gauge. In subsequent gauges considerable changes have been made in design to cheapen construction and to improve waterproofing.

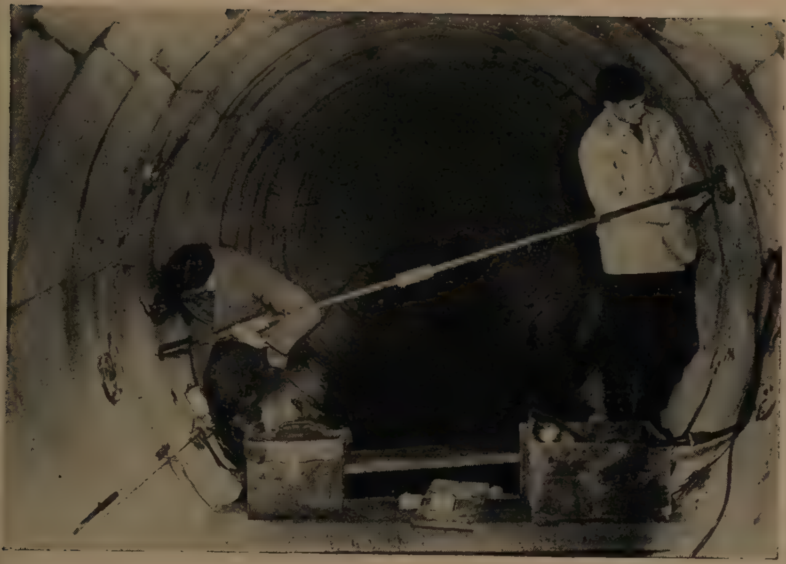


FIG. 5.—MICROMETER ROD FOR MEASURING CHANGES IN DIAMETER OF THE LINING

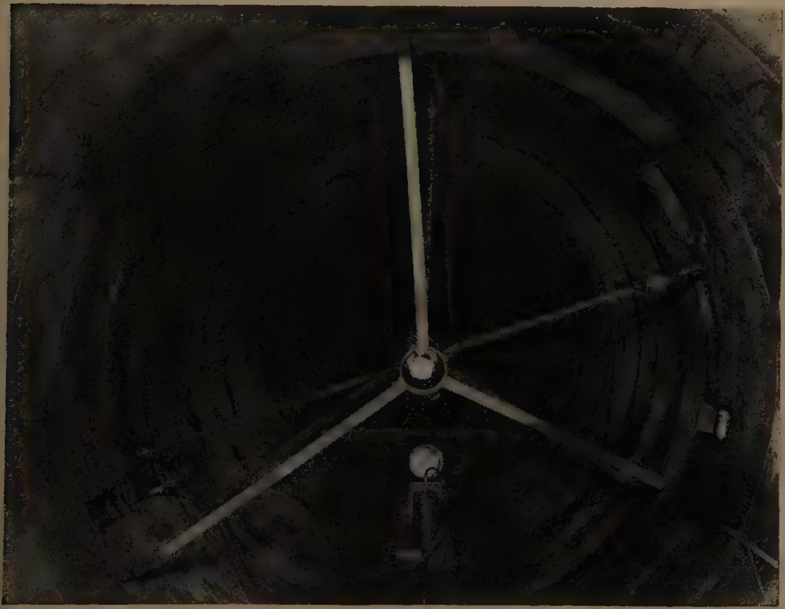


FIG. 15.—GENERAL VIEW OF HOOP-LOADING TEST ON RING NO. 981

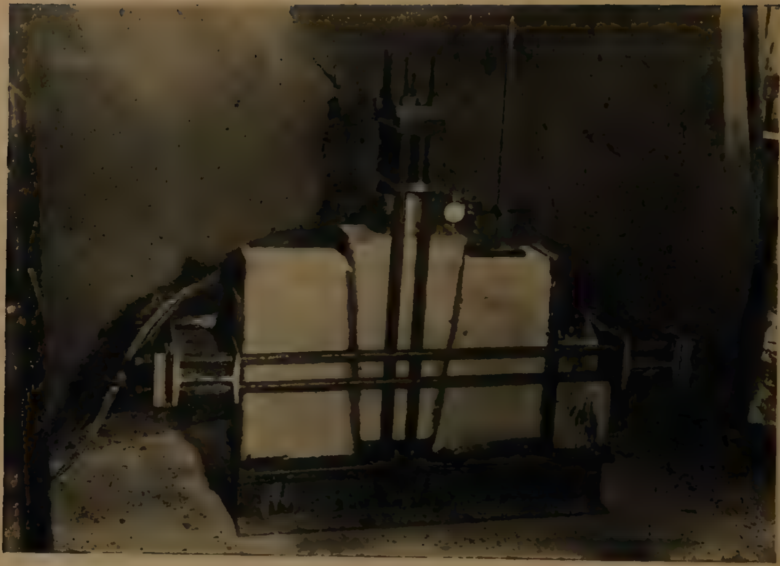


Fig. 19.—Apparatus for carrying out tests on model segments

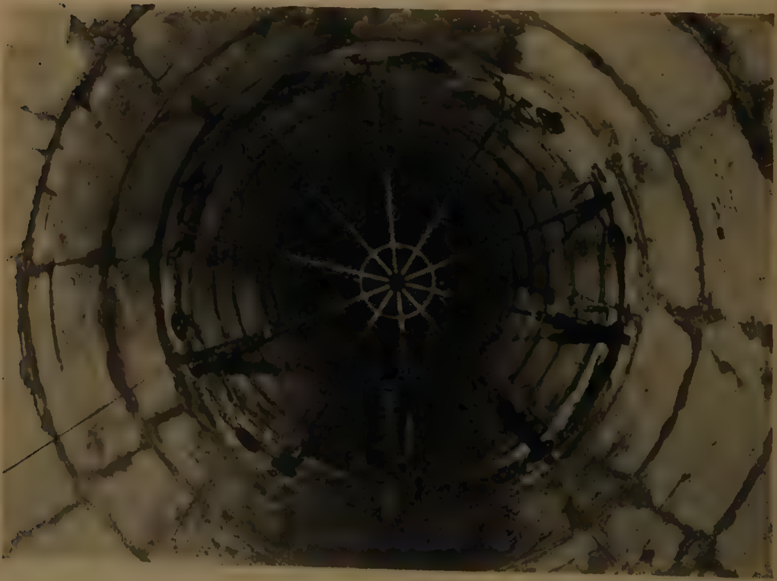


Fig. 24.—General view of instruments installed in tunnel at Ring No. 352
showing diameter gauges, vibrating-wire radial pressure gauges, and
joint movement gauges

A bronze sleeve with an inner flange, made from the same mould used for the hydraulic gauge cylinder, was cast in the centre of the concrete segment (see Fig. 3). A steel load plate 8-in. square, machined to the curvature of the segment, bears on the clay and on one end of the vibrating-wire load gauge. The other end of the load gauge bears on a cover plate bolted to the flange of the bronze sleeve. There is a good clearance between the load plate and the concrete and between the load-plate guide, the bronze sleeve, and the load gauge.

This load gauge is 5 in. long, has a diameter of $2\frac{1}{2}$ in., and $\frac{1}{8}$-in. walls. The vibrating wire is silver-plated piano wire, 0·018-in. dia. and effective length 4 in. Its calibration is good up to 3 tons and it can be read confidently to the nearest 0·02 ton. The pressure on the load plate can be measured to the nearest 0·04 ton/sq. ft.

When waterproofed the load gauges were calibrated under varying external water pressures. The effect of this pressure was found to be equal to the total water load on the end area of the cylinder, and equivalent to only 0·18 ton/sq. ft in pressure on the load plate for 100-ft head of water.

Vibrating-wire load gauges to measure hoop thrusts in segments

After studying the behaviour of a large number of rings in the course of erection, it was realized that load gauges could be safely erected to measure the hoop thrust. It was evident, also, that load gauges could be used more effectively and economically in this position than as radial pressure gauges.

A gap equal in width to the height of a load-gauge cylinder (5 in.) was formed in a segment by casting the concrete on either side of a pair of 1-in.-thick steel plates suitably spaced and mounted in the normal segment mould. This produced a " split " segment with two parallel steel faces in the centre, which were held apart by a pair of load gauges when erected in the tunnel (see Fig. 4). For reasons of stability during erection of the ring, the split segments were placed only in the lower half of the tunnel section, and in the positions of segments not subject to longitudinal sliding from the shield jacks.

The load-gauge cylinders used in this position could carry loads up to 40 tons and one pair was installed at each split segment. The cylinders were 5 in. long, with a wall thickness of about $\frac{1}{2}$ in., and an outside diameter of $3\frac{1}{2}$ in., and could be read to the nearest 0·1 ton. The effect of the water head on these load gauges was practically negligible.

Diameter-measuring rod

The diameter-measuring rod was used to measure the distance between diametrically opposite pairs of segments whenever the tunnel was empty. A view of the rod in use is shown in Fig. 5; it consists very simply of a length of $\frac{3}{4}$-in. steel gas-pipe with a 0·001-in. screw micrometer

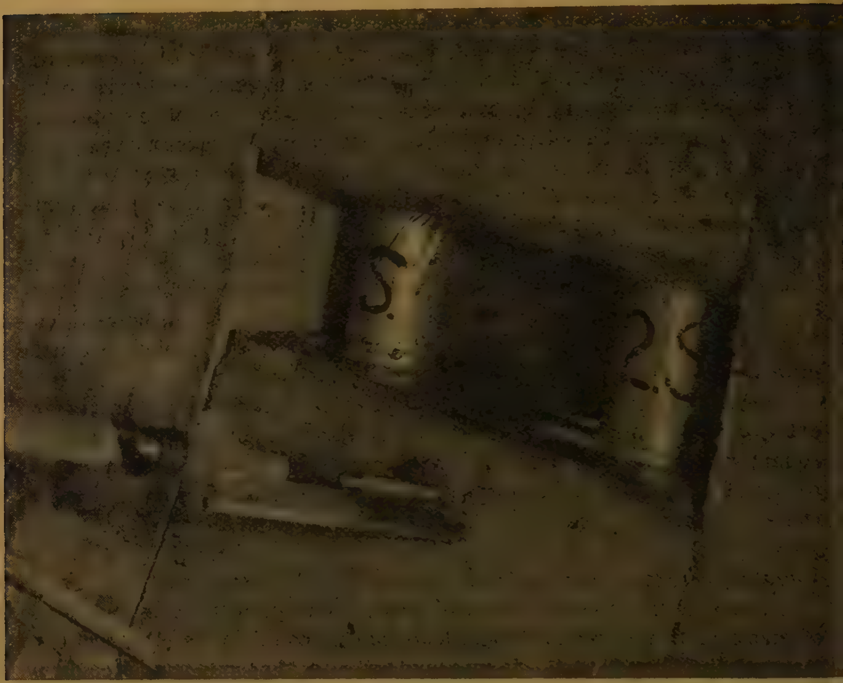


FIG. 4.— A PAIR OF LOAD GAUGES INSTALLED TO MEASURE HOOP THRUST
(The micrometer head of the diameter-measuring rod can also be seen)

at one end and a small hard steel flat surface at the other. Handholds on the rod were heat insulated. In case of accidents to this rod, a similar length of 2-in. × 2-in. channel with hemispherical-ended pegs mounted on brackets at each end, was kept in the tunnel as a reference length. Rustless dome-shaped screws were set in every segment at each experimental ring.

Position of experimental rings and method of installing instruments

The positions of the experimental rings with the numbers and types of load measuring instruments at each ring are shown in Fig. 1 and Table 1. The positions of these rings along the tunnel were not fixed specifically. When a group of instruments was ready, they were installed at the first convenient opportunity. Tunnel construction started at ring No. 1 at the west shaft on the 7th January, 1952, and proceeded without interruption to the east shaft on the 20th May of the same year.

Throughout the Paper, unless otherwise stated, the segments of each ring are numbered from the soffit segment as No. 1, in a clockwise direction looking towards the shield, as shown in Fig. 1.

In the installation of each experimental ring the aim was to reproduce the normal construction conditions as nearly as possible. Details of the methods of installing the various instruments at each ring are given below.

Rings 116–117

Ten hydraulic radial pressure gauges were fitted in alternate segments in these rings on the 26th January, 1952, and were located in segments 1, 3, 5, 7, 9 of ring 116 and in segments 2, 4, 6, 8, 10 in ring 117.

Rings 179–180

Ten vibrating-wire radial pressure gauges were fitted in alternate segments of these rings on the 4th February, 1952 and in every respect their positions in these two rings are a replica of those in rings 116–117. The gauged segments were not subjected to direct thrusts from the shield rams. The steel load plate for each gauge was held flush with the outer surface of the concrete during the initial erection of the ring by a spacing device. After all but the last two segments had been driven, the spacers were removed and the vibrating-wire load gauges put in their place. The outer flange bolts were tightened gently and evenly by hand, so that the load gauge was bedded down evenly at each end with a very small load in it. The ring was then driven into its final position by the shield rams thrusting on the remaining segments in the usual way. The loads in the gauges built up immediately and the load increase was usually followed on one gauge while jacking was in progress. Frequent readings were taken on all gauges for the first few hours after jacking, when subsequent rings were being erected, and then daily for the next few weeks.

Ring 352

One vibrating-wire radial pressure gauge was fitted to every one of the ten segments in ring 352 ; hence half the gauges, 1, 3, 5, 7, 9 were in segments which were pushed directly by the shield rams. The sliding of the odd-numbered segments against the clay does not appear to have caused jamming of the load plates against the concrete. A band of claystone was noted in the vicinity of segments 3 and 4 at ring 352. This ring was erected on the 22nd February, 1952.

Rings 755–756

Two pairs of vibrating-wire load gauges were inserted in split segments 4 and 6 of ring 755, and another two pairs inserted in split segments 6 and 8 of ring 756 to measure the hoop thrust. Hydraulic radial pressure gauges were inserted in segments 1, 3, 5, 7, 9 of ring 755 and in segments 2, 4, 5, 7, 10 of ring 756. A number of these gauges had troublesome leaks in the bronze castings and were expected to prove difficult in use. Segments 1, 3, 5, 7, 9 were pushed by the shield rams in both these rings. These rings were erected on the 10th April, 1952.

Rings 980–982 and 992–994

These are two groups of three similar experimental rings. The central ring of each group, namely 981 and 993, contained a split segment with a pair of hoop load gauges at position 4. In every ring a split segment with a pair of spacers was inserted in position 8. The spacers permitted the subsequent insertion of a pair of special jacks for loading the rings circumferentially. The rings were erected on the 12–13th May, 1952.

Rings 981 and 993–1,021

In segment 3 in each of these rings, two 2-in.-dia. holes were formed through the segment. The holes were treated in three ways :—

(1) Completely unsealed so that the clay was in free contact with the tunnel atmosphere.
(2) Fitted with screw plugs at the inner face, but the clay left unconstrained.
(3) Fitted with a wooden plug behind the screw plug to press on the clay.

These holes are for the purpose of boring into and sampling the clay behind the lining at long intervals after construction.

Tunnel atmosphere and water conditions

The amount of heat energy released in the tunnel during construction was very small and, until the east shaft was reached on the 20th May, there was no through ventilation from outside. The mean tunnel air temperature increased gradually from 52 to 54°F and the relative humidity was about 97%, which was sufficient to cause drying of isolated samples of clay in the tunnel. Water condensed on the roof segments and the release of compressed air at the shield pump often produced a thick fog. In spite of condensation it was evident that, within about one month of erecting a segment, water from the clay began to appear in places on the roof segments and run down the sides of the lining.

As soon as the tunnel broke through into the east shaft a noticeable draught was created, the relative humidity dropped that day to 94% and in 4 months was down to 80% with a temperature of 57°F ; subsequently, with the onset of winter, the temperature fell to 52°F. During the ventilated period, water was quite clearly seeping into the tunnel through the clay, at some points more than others. The rate of inflow was minute and only sufficient in the worst places to maintain a glossy wet film down the lining periphery. There is evidence of water coming out of the clay in another poorly ventilated tunnel at the same depth in central London.

Observations on the hydraulic radial pressure gauges in rings 116, 117, 755, and 756

These proved the least satisfactory of all instruments installed on account of unavoidable leaks, delays in final installation, and the trial-and-error method of observation. It was not possible to follow the increase in load during and soon after jacking the segments with these instruments.

The typical performance of a gauge (ring 116, segment 9) during a long period when the applied pressure was steadily increased and decreased is shown in Fig. 6. The instrument was read daily and any pressure change occurring overnight was corrected. The inward movement of the piston towards the tunnel was not halted until the applied pressure reached about 1·9 ton/sq. ft. An attempt was then made to push the piston back to its initial position and it will be noted that quite high pressures were recorded; the piston could not be pushed back completely without a risk of bursting the bellows capsule. As the pressure was reduced the piston again moved inwards, but it did not move rapidly until the pressure was reduced below about 1·9 ton/sq. ft. The pressure was only built-up to the extent shown in Fig. 6 on a few gauges; others were kept as near the balance point as possible. Because of the load-deflexion characteristics of the piston it is difficult to interpret the lining pressure equivalent to no movement of the piston at the time of jacking the ring. The average radial pressure on the lining at each pair of rings, assuming the lining

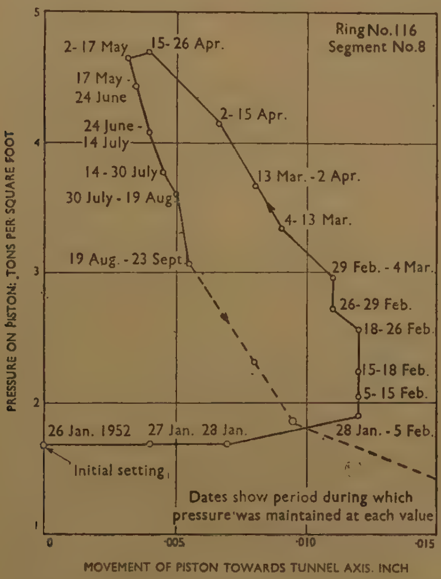


FIG. 6.—TYPICAL PERFORMANCE OF A HYDRAULIC EARTH-PRESSURE GAUGE

pressure to be equivalent to the bellows pressure which first brought the piston to rest, is :—

Rings	Average radial lining pressure
116/117	2·2 tons/sq. ft
755/756	2·1 „ „ „

The use of the gauges was not continued after the end of 1952 because the tunnel was soon to be flooded.

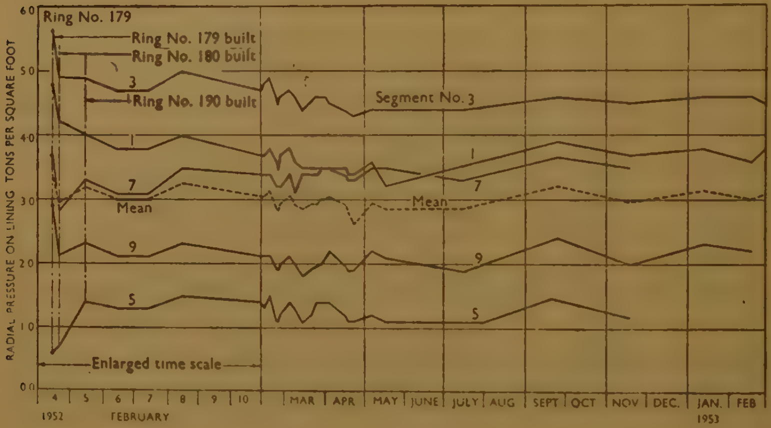


FIG. 7.—VARIATION IN RADIAL PRESSURE : RING NO. 179
(Vibrating-wire radial pressure gauges in alternate segments)

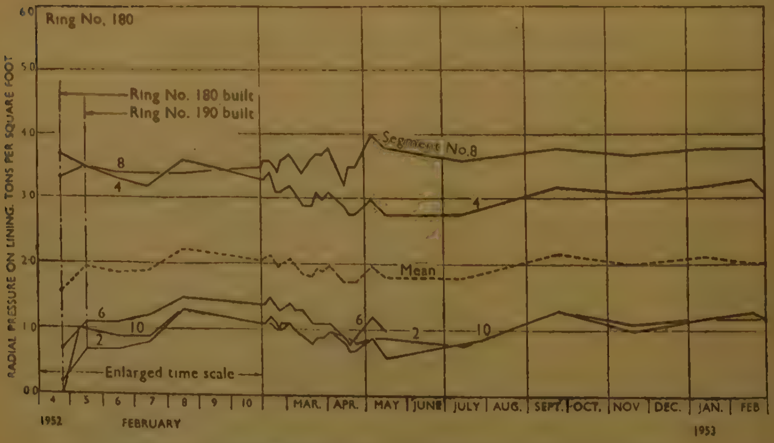


FIG. 8.—VARIATION IN RADIAL PRESSURE : RING NO. 180
(Vibrating-wire radial pressure gauges in alternate segments)

Observations on vibrating-wire radial pressure gauges

The observations of pressure on each gauge for rings 179, 180, and 352 are plotted with time in Figs 7, 8, and 9, an enlarged time scale being used for the first week of observations. Some of the erratic pressure variations with time were probably caused by instability of the reference gauge because it was not until the end of 1952 that the procedure of checking each reading against a stable tuning fork was adopted. The pressures built up immediately in the course of jacking the ring and while the next few rings were built there were noticeable interchanges of load. There is a considerable scatter in the pressure values, but apart from segments 3 and 4 of ring 352 where a claystone existed in the vicinity, the pressures on individual segments range between about 1 and 5 tons/sq. ft in each ring. The mean value of the pressure acting on each ring is plotted as a dotted line. The rough values subsequent to 2 months after erection are 3·0, 2·0, 4·2 tons/sq. ft in rings 179, 180, and 352 respectively. There appears to be

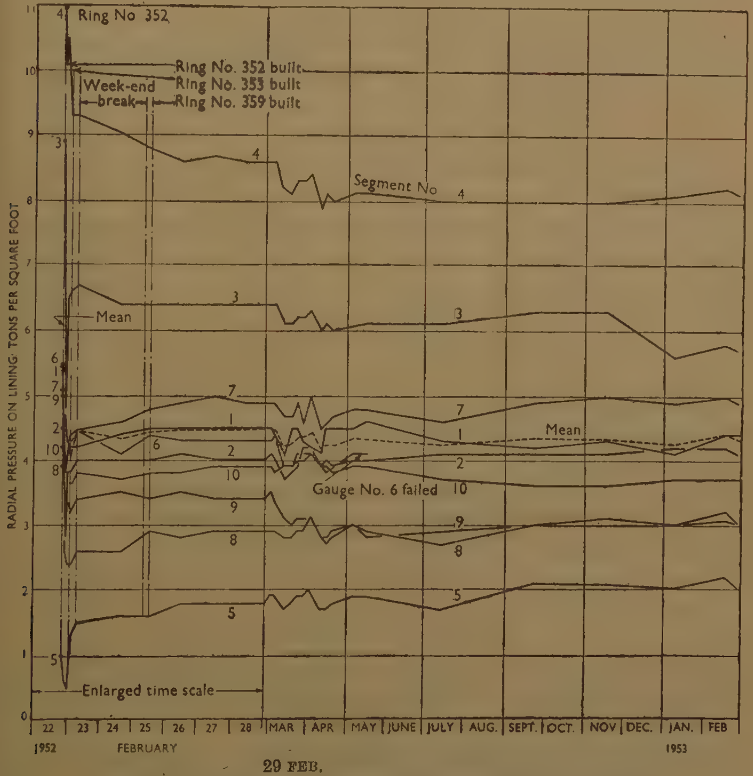


FIG. 9.—VARIATION IN RADIAL PRESSURE: RING NO. 352
(Vibrating-wire radial pressure gauge in every segment)

a slight increase in pressure amounting to about 5% towards the end of the year. Four of the gauges broke down during the year, two from the electrical insulation breaking in awkward places, and two from damaged leads and flooding by water standing in the invert of the tunnel.

Observations on hoop-thrust gauges

The hoop-thrust measurements are plotted in Figs 10 and 11 for rings 755 and 756. Each curve represents the sum of the loads in the pair

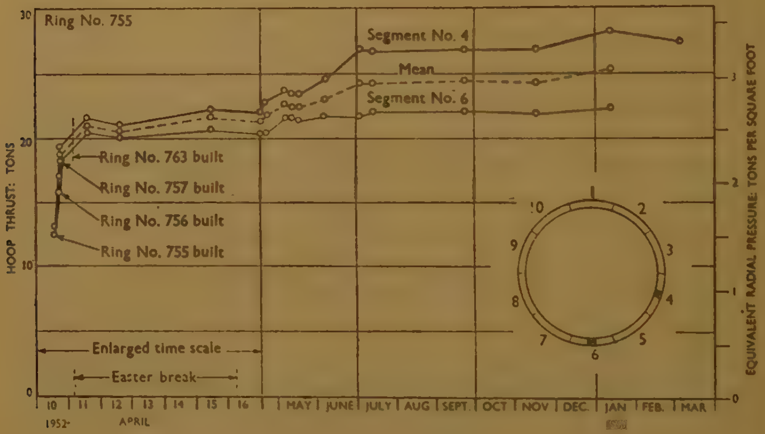


FIG. 10.—VARIATION IN HOOP THRUST: RING NO. 755
(Hoop-thrust gauges in segments Nos 4 and 6)

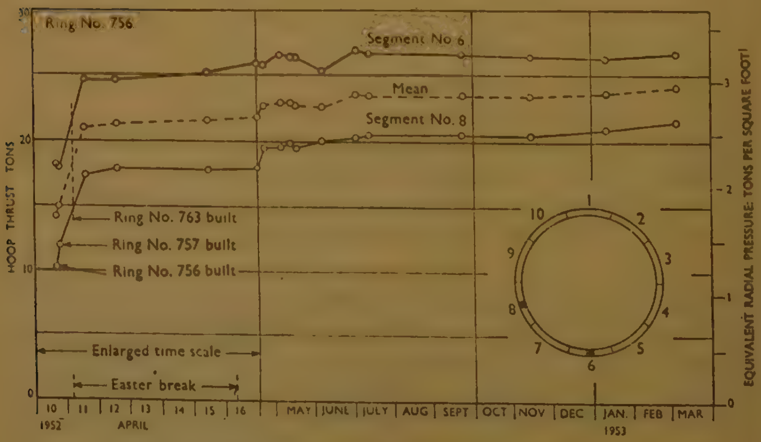


FIG. 11.—VARIATION IN HOOP THRUST: RING NO. 756
(Hoop-thrust gauges in segments Nos 6 and 8)

of thrust gauges at each segment. Again the loads built up immediately on jacking of the ring. The loads increased in both rings with the construction of subsequent rings, except for a quiescent period during the Easter vacation. There is a small increase in load during the rest of the year. Segment 9 in ring 755 and segment 3 in ring 756 were the last segments to be rammed into position. It will be noted that the thrust varies around the ring.

If the average thrust in the ring can be assumed to be equal to the mean of the two observed values, then the equivalent mean radial pressure is about 3 tons/sq. ft in ring 755 and 2·8 tons/sq. ft in ring 756 subsequent to about 2 months after erection. These values are given merely for the purpose of making a rough comparison with the measured radial pressures in these and the other experimental rings.

Observations of relative changes in diameter of the lining

The changes in diameter between segments 1–6, 2–7, 3–8, 4–9, and 5–10 for rings 179, 180, and 352 are plotted with time in Figs 12, 13, and 14 and they are typical of the changes in the other experimental rings. They are plotted from an origin at a date corresponding to the time of construction about twenty rings ahead of the ring under consideration. The early movements, which are rather erratic owing to the influence of local jacking, are tabulated in Figs 12, 13, and 14.

Results from all rings show that there has been a very small and gradual change in diameter until early in 1953. The change is symmetrical on a diameter about halfway between diameters 1–6 and 2–7 (i.e., nearly vertical) where there is a maximum shortening of diameter of about 0·06 in. There is a maximum increase in diameter of about 0·02 in. on the orthogonal diameter. The circumference on the lining has decreased in length by about 0·05 in. It is not known, in the absence of absolute measurements, whether the change in shape is symmetrical about the tunnel axis.

The distances between diametrically opposite segments when the rings were built are given in the Tables in Figs 12, 13, and 14. At each ring the vertical diameters are greater than the horizontal ones by about 0·5 in.

It may be noted that the direction of the largest diameter (vertical) practically coincides with the greatest subsequent decrease in diameter, but there is no evidence from the load observations to associate these facts.

After March 1953 the diameter changes plotted relate to the values recorded before each filling and after each emptying of the tunnel and are discussed later in the Paper.

Observations of water-content changes in the clay behind the lining

Water-content determinations have been made on samples of clay from borings through the 2-in.-dia. holes in the lining at various times up to 30 months after local construction. These determinations have been made at intervals of $\frac{1}{8}$ or $\frac{1}{4}$ in. for various distances behind the outside

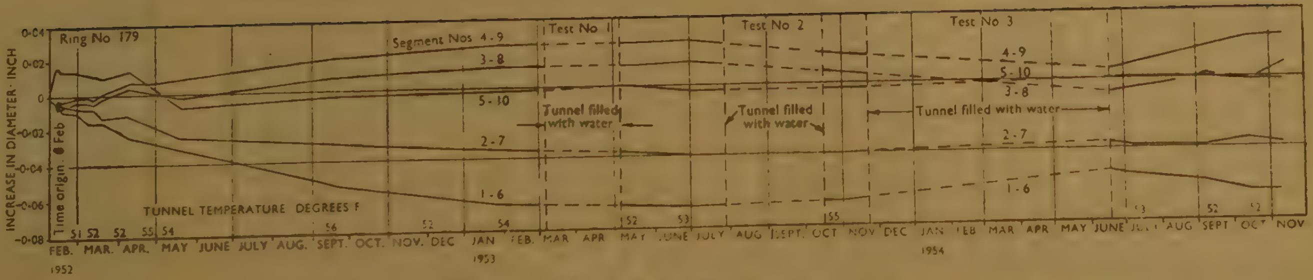


FIG. 12.—CHANGES IN DIAMETER: RING NO. 179

(Ring No. 179 built 4 Feb., 1952. Changes in diameter are measured from a zero reading on 6 Feb., 1952, when Ring No. 200 was built)

Segment Nos	1–6	2–7	3–8	4–9	5–10
Internal dia. after building ring on 4.2.52: in. . . .	100·31	99·87	99·78	99·66	99·91
Increase in dia. between 4 and 6 Feb., 1952: in. . . .	+ 0·006	− 0·031	− 0·027	− 0·034	− 0·005

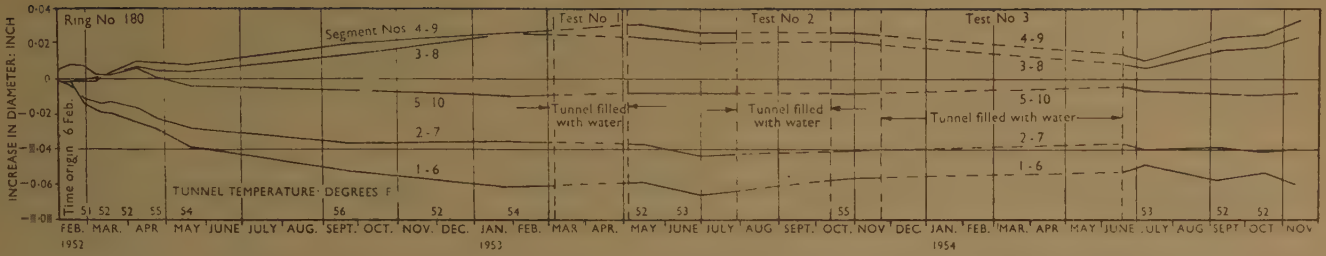


FIG. 13.—CHANGES IN DIAMETER: RING NO. 180

(Ring No. 180 built 4 Feb., 1952. Changes in diameter are measured from a zero reading on 6 Feb., 1952, when Ring No. 200 was built)

Segment Nos	1–6	2–7	3–8	4–9	5–10
Internal dia. after building ring on 4.2.52 : in. . . .	100·31	99·88	99·73	99·66	99·86
Increase in dia. between 4 and 6 Feb., 1952 : in. . . .	+ 0·050	+ 0·017	+ 0·011	+ 0·013	+ 0·018

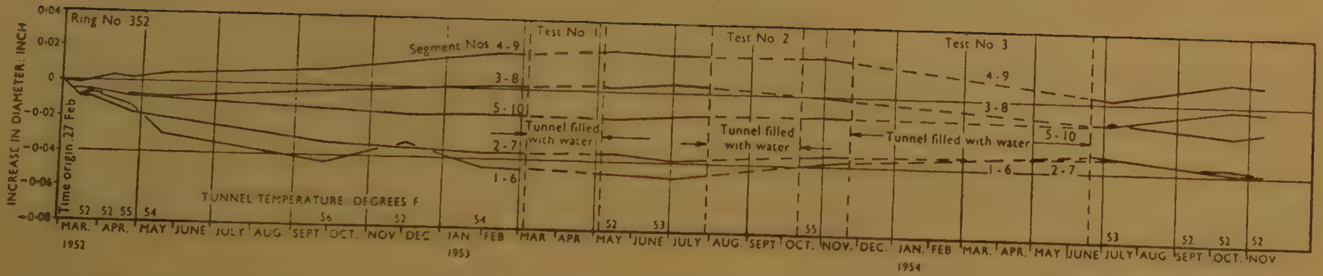


FIG. 14.—CHANGES IN DIAMETER: RING NO. 352

(Ring No. 352 built 22 Feb., 1952. Changes in diameter are measured from a zero reading on 27 Feb., 1952 when Ring No. 372 was built)

Segment Nos	1–6	2–7	3–8	4–9	5–10
Internal dia. after building ring on 22.2.52: in. . . .	100·27	99·99	99·67	99·72	99·90
Increase in dia. between 22 and 27 Feb, 1952: in. . . .	+ 0·007	− 0·034	− 0·092	− 0·064	− 0·018

face of the lining. The water content has increased and the clay has softened to a depth of not more than about $\frac{3}{4}$ in. The softening is associated with the presence of the hole and the depth of softening is a function of the size of the hole, as might be expected. Behind one $3\frac{1}{2}$-in. drain hole the softening proceeded to a depth of 3 in. Nearly all the softening occurred in the first fortnight or 3 weeks after local construction. The clay initially at a water content of about 25% increased to about 30% and, in a few cases, to more than 40% in the first $\frac{1}{8}$ in. from the lining. The different types of fittings in the lining holes had little effect; the type fitted with a wooden plug reduced the amount of softening to some degree.

There is no indication yet from water-content measurements of any deeper softening and the softening behind the intact segments is probably no more than skin-deep. In places where the clay is not in contact with the segments, local softening will proceed to a depth determined by the size of the gap.

Summary and discussion of observations

The average values of the radial pressure on the lining (or its approximate equivalent in the case of hoop-thrust measurements) immediately after erection and after about 2 months are summarized in Table 4, together with details of the last driven segment which closed the ring.

TABLE 4

Ring No.	Radial pressure on lining: tons/sq. ft		Last driven segment	
	Immediately after erection	After 2 months	Position (segment No.)	Pressure in 7-in.-dia. ram of shield. lb/sq. in.
116/117	—	2·2	—	—
179	3·4	3·0	—	—
180	1·6	2·0	3	2,000
352	5·4	4·2	—	2,000
755	1·6	3·0	9	1,200
756	1·7	2·8	3	1,800
981	0·9	2·2	5	1,200
993	2·6	2·8	5	2,000

Notes:—1. The values for rings 116/117 are from hydraulic gauge observations.
2. For rings 755 and 756, only the values for vibrating-wire-gauge observations are given.

It will be seen that observations by vibrating-wire radial pressure and hoop-thrust gauges are in reasonable agreement, and the Authors consider that the most effective way of investigating loads on this type of tunnel lining is to insert hoop-thrust gauges.

There is an indication that the lining pressures increased by about 5% towards the end of the first year after construction.

The appreciable scatter in the lining pressures recorded on the individual load plates of the vibrating-wire gauges may not be much greater than the variations of pressure on parts of any actual ring. The segments did not bear regularly on the clay before jacking and in these circumstances the load plate is likely to have been set protruding in some cases.

The most significant result is that practically all the pressure on the lining arises from the initial jacking force. The only possible indication of an earth-pressure effect as normally understood is the slight increase in pressure towards the end of the first year. It might indeed be expected that little earth pressure would develop because of the high compression strength of the clay (300 lb/sq. in. average). Simple elastic theory of a weighted homogeneous medium shows that even if the clay received no support from the lining whatsoever, the maximum shear stress in the clay would be not more than 80 lb/sq. in., or only about half the strength of the clay. In these circumstances, the stress on the lining depends almost entirely on the conditions during the erection of the individual rings.

A complete understanding of the changes in size and shape of the shield, of the unlined hole, and the lined hole has not been obtained, chiefly on account of difficulties of taking detailed observations at the shield itself. The shield is a difficult object to measure, but when it was taken out of the tunnel it was practically circular and certainly not elliptical to the extent of 0·5 in. like the finished tunnel lining. The elliptical shape of the finished lining may arise from one or both of two causes :—(*a*) the shield may become elliptical itself in travelling through the clay ; (*b*) the clay may be more compressible in the vertical than in the horizontal direction and thus the lining would expand more in the vertical direction in the course of jacking. The Authors consider that the second cause is more likely because the clay is slightly laminated in the horizontal plane. The very small change in shape of the lining after construction, i.e., a decrease of 0·06-in. dia. vertically and an increase in diameter of 0·02 in. horizontally was not associated with any measurable change in the distribution of lining pressure. The net decrease in circumference of the tunnel by 0·05 in. probably results from extrusion of bitumen paint at the joints.

Any change in the condition of the clay behind the lining associated with construction cannot be detected by water-content measurement, but this is not a sensitive method of measuring changed conditions in an over-consolidated clay.

INVESTIGATIONS INTO THE MECHANICS OF ERECTING THE LINING

Observations made during the period when the tunnel was empty indicated that the London Clay, owing to its high strength, was capable of sustaining a wide variation of pressures imposed on it by the tunnel lining.

The magnitudes of these pressures appeared to be largely determined by the conditions during the erection of individual rings of segments.

It was noticed when tightening the rings that, although driven in pairs, one segment was always the last to be jacked home. This segment is often referred to in the Paper as the key segment, and it was during its movement that the jack thrust, and also the pressures on the lining, built up noticeably. It was also clear that there was considerable friction between adjacent segments, and between the backs of the segments and the clay.

Hoop-loading test on tunnel

The final tightening process, apart from the wedge action of the last driven segment itself, was reproduced in a circumferential loading test on a special section of the tunnel between rings 980 and 982, described on p. 18. A pair of hoop-thrust gauges were installed in segment 4 of ring 981 similar to those at rings 755 and 756 (Fig. 4). Split segments were similarly inserted at position 8 of each of the three rings, the bearing plates being supported by spacing pieces, so that a pair of special 50-ton hydraulic jacks could be inserted in any ring to apply a circumferential thrust.

Chromium-plated reference studs were fixed to the centre of each segment, and absolute radial movement was measured by a diametrical arm, which rotated on a shaft set up on the axis of the tunnel, and carried a screw micrometer at each end. The shaft was supported at its ends from segments clear of the test rings. A general view of this apparatus is shown in Fig. 15, facing p. 412.

Tests were carried out with the pair of jacks inserted in the centre ring (981), load-gauge thrusts and deformations being observed for each increment of jack thrust. The jack thrust required just to unload the spacing pieces supporting the bearing plates gave an indication of the initial thrust in segment 8, which was then increased in stages up to a maximum of 99 tons where it was maintained for about half an hour. The ring was subsequently unloaded in similar stages and the time occupied by a complete test was about 3 hours.

Fig. 16 shows the relation between the jack thrust at segment 8 and the hoop-gauge thrust at segment 4 during the first two loading cycles. The effect of friction, most of which was between the backs of the segments and the clay, can be clearly seen. A jack thrust of about 60 tons was needed to overcome the initial friction, and at a maximum thrust of 99 tons the hoop-gauge reading at an angular distance of 144° was only 35 tons. After the jack thrust had been maintained at 99 tons for half an hour, the hoop-gauge reading had only increased by about 2 tons and the rate of any further increase was very small.

During unloading it was found that the jack thrust could be reduced to zero, whilst the thrust in segment 4 had only fallen to 27 tons. In this condition the ring was being held in place entirely by frictional and adhesive forces and, after 20 minutes, the distance between the jack bearing plates of segment 8 had decreased by only 0·01 in. with no significant change in the hoop-gauge reading.

To prevent any further movement, the jack bearing plates were then supported on adjustable spacing pieces and by the time the next test was carried out in 3 to 4 weeks the thrusts in segments 4 and 8 had altered to approximately their original values.

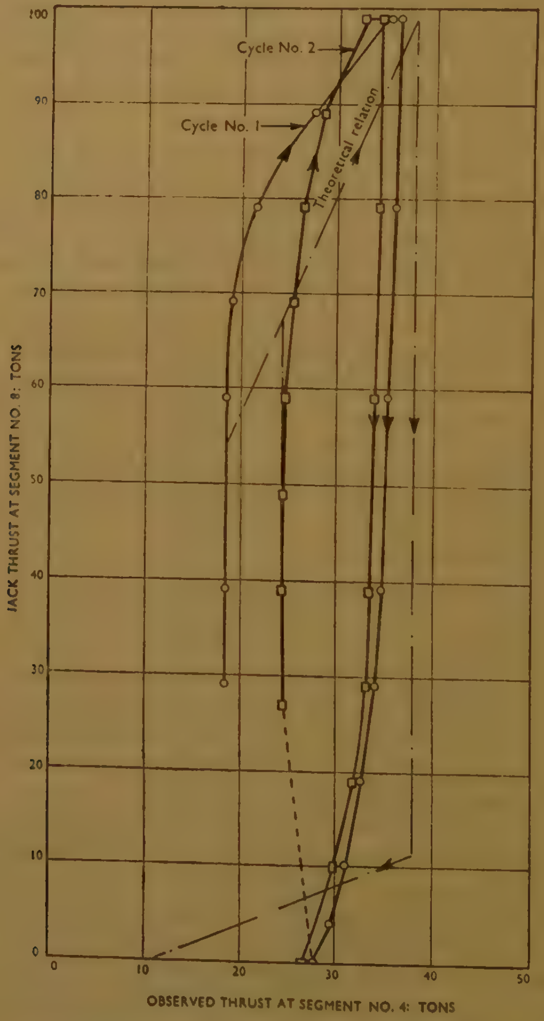


FIG. 16.—HOOP LOADING TEST ON RING NO. 981 : RELATION BETWEEN THRUSTS

Fig. 17 shows the radial movements of individual segments for the second loading cycle. To obtain a rapid indication of deformation changes, the distance between the jack bearing plates was also recorded. These latter observations were converted to an equivalent mean change in radius and are also plotted in Fig. 17 where it will be seen that they are in reasonable agreement with the mean of the direct radial observations. It is also to be noted that for any given jack thrust the radial movements of individual segments vary from a maximum at the jacks to almost zero at the opposite end of the diameter where the change in hoop thrust is very small. While subjecting the ring to a jack thrust of 99 tons, the mean increase in

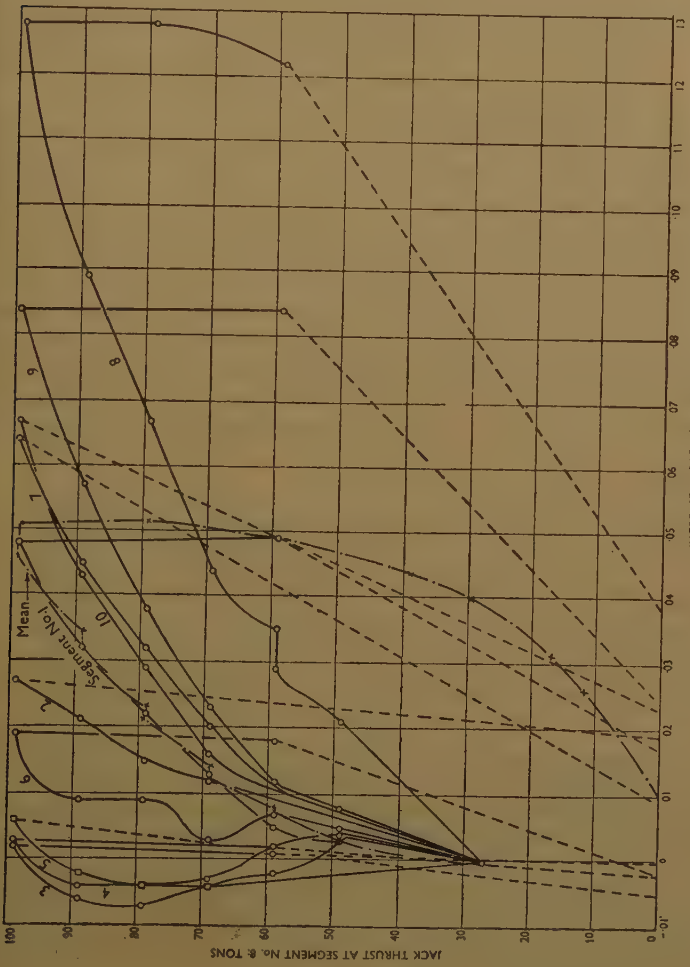


FIG. 17.—HOOP LOADING TEST ON RING NO. 981. RADIAL MOVEMENTS OF INDIVIDUAL SEGMENTS: CYCLE 2

radius was 0·005 in. during half an hour, and the rate of any further increase was then very small.

Applying a circumferential thrust to the outer rings was found to have little significant effect on the behaviour of the central ring.

Laboratory determination of the friction between clay and concrete and between two concrete surfaces

The hoop-loading tests clearly showed the great influence of friction in determining the pressures on the tunnel lining. In order to analyse the mechanics of tightening a ring it was therefore necessary to have direct measurements of these frictional stresses.

The frictional stress between a clay-concrete interface was measured by a series of tests in 6-cm and 12-in.-square shear-boxes. The concrete specimen was cast in the bottom half of the box with its surface finished as smoothly as possible flush with the shear plane of the box. The London Clay was obtained from excavations at about 70 ft below surface and was generally similar to that encountered in the tunnel.

Tests were carried out for three conditions of the interface, namely, dry, lubricated with water, and lubricated with soft soap, the results being shown in Fig. 18. The last two conditions are part of an investigation into means of reducing friction, and are discussed in a later section of the Paper.

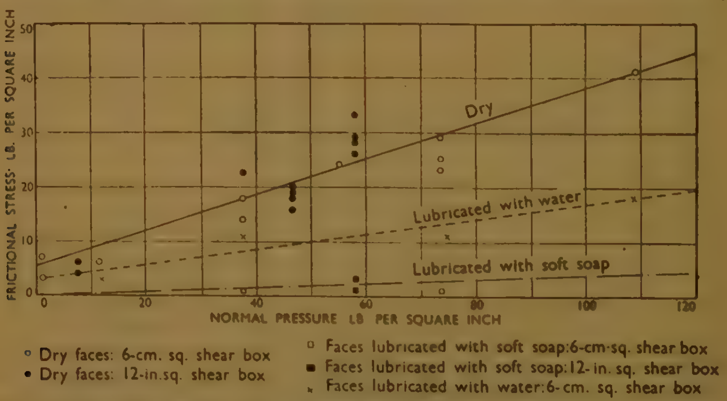


FIG. 18.—RESULTS OF SHEAR-BOX TESTS TO DETERMINE THE FRICTION BETWEEN SMOOTH CONCRETE AND A SMOOTH FACE OF STIFF LONDON CLAY

Within the limits of experimental variation, the straight lines plotted are considered to give a reasonable mean estimate of the frictional stresses. The equations of these lines are :—

Dry $s = 5 + 0{\cdot}34n$ lb/sq. in.
Wet $s = 3{\cdot}5 + 0{\cdot}13n$ „ „
Soft soap $s = 0{\cdot}04n$ „ „

where s denotes the frictional shear and n the normal pressure.

The frictional forces between the concrete sliding surfaces were determined from a concrete model of the key and its adjacent segments (Fig. 19, facing p. 413). The three segments were each 21 in. × 6 in. × 10 in. mean width, the 21-in. dimension of the model corresponding to the 21-in. length of a tunnel ring, and were plane instead of curved to the radius of the tunnel. The central key segment was tapered 1 : 10 on each side as in the tunnel, whilst the outer segments were tapered on one side only. By means of hydraulic jacks in a loading frame the longitudinal thrust required to move the key against a range of transverse thrusts applied to the outer segments was measured.

A series of tests was carried out on nominally identical sets of model segments, using a dried bitumen paint film on the longitudinal sliding surfaces of all three segments, and on the transverse bearing surfaces of the outer segments.

A wide variation in results was obtained and the most influential cause was traced to small variations in the porosity of the concrete surfaces, which varied the amount by which the paint film became pressed into the pores of the concrete. When a tested paint film was cleaned off with petrol and replaced by a fresh one, the repeat test almost invariably gave a lower angle of friction. A double coat of paint also reduced the angle of friction but normally a thin film, equivalent to that used in the tunnel, was applied. The experimental values for the angle of friction are as follows :—

Thin film of bitumen paint on concrete.

New surfaces	13° and 16·5°
Surfaces cleaned off and repainted .	5°, 5°, 5°, and 12°

Unlubricated dry concrete.

New surfaces	22°

Since the concrete of the models was in all cases less dense than that of the tunnel segments the angle of friction for concrete lubricated with bitumen paint probably lies somewhere between the values given above.

Theoretical analysis

While the key segment was being driven home to tighten the ring, the segments on either side of the key were displaced circumferentially. This movement induced opposing frictional forces which caused the hoop thrust to decrease as the distance from the key increased. Ideally, the displacements and thrust distribution should be symmetrical about the diameter through the keying segment, the thrust decreasing to a minimum at the remote end of the diameter. Owing to badly fitting joints, this distribution must, however, often become distorted. For these reasons, a rigorous theoretical analysis is impossible but, by making a number of simplifying assumptions, it is possible to predict the general trend of pressure distribution in terms of the thrust required to drive home the key segment.

Assumptions

(1) The key segment alone is subjected to the ram thrust applied by the shield, the rest of the ring having already been driven home.

(2) Displacements and pressure distributions around the ring are symmetrical about the diameter through the keying segment.

(3) The thrusts between the longitudinal joints act in planes tangential to the tunnel surface. Their directions in these planes are shown in Fig. 20.

(4) The reaction to the longitudinal thrust of the key segment is taken entirely on the transverse joints of the two segments adjacent to the key.

(5) The full frictional resistance is developed on all the sliding surfaces.

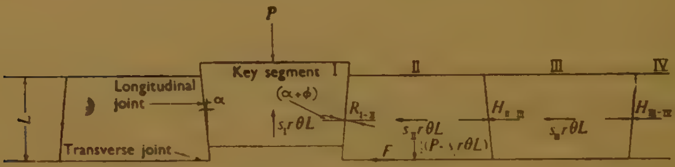


FIG. 20.—THEORETICAL ANALYSIS OF THE MECHANICS OF ERECTING A RING

For the purpose of this analysis only, the segments are indicated by Roman numerals from the key segment as No. I. A numerical suffix to a symbol indicates the segment or joint to which it applies, and the significance of the symbols is shown in Fig. 20.

e denotes average radial pressure acting on one segment.
s ,, frictional shear stress developed on the clay/concrete interface and :—
$s = A + Be$, where A and B are constants.
p ,, pressure in cylinder of 7-in.-dia. ram of shield.
P ,, thrust applied by ram.
R ,, thrust between the longitudinal joints of the key segment.
H ,, thrust between the longitudinal joints of the remaining segments.
ϕ ,, angle of friction on longitudinal joints painted with bitumen paint.
ϕ' ,, angle of friction between bare concrete joints.
r ,, outside radius of ring.
L ,, length of ring measured axially along the tunnel.
θ ,, mean angle subtended by segment at centre of ring.
α ,, inclination of longitudinal joint with a line parallel to the axis of the tunnel.

Considering the forces acting on the key segment, it can be shown that for limiting equilibrium :—

$H_{I/II}$ the component of $R_{I/II}$ in a plane perpendicular to the axis of the tunnel is given by :

$$H_{I/II} = \frac{P - s_I r\theta L}{2\cos\frac{\theta}{2}\tan(\alpha + \phi)}$$

Since

$$s_I = A + Be_I \text{ and } e_I = \frac{H_{I/II}}{rL},$$

$$e_I = \frac{P - Ar\theta L}{2rL\cos\frac{\theta}{2}\tan(\alpha+\phi) + Br\theta L} \quad . \quad . \quad . \quad . \quad . \quad (1)$$

and

$$H_{I/II} = \frac{P - r\theta L(A + Be_I)}{2\cos\frac{\theta}{2}\tan(\alpha+\phi)} \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Considering the forces acting on segment II, the friction on the transverse face is given by :

$$F = \tfrac{1}{2}(P - s_I r\theta L)\tan\phi' \quad . \quad . \quad . \quad . \quad . \quad (3)$$

and for limiting equilibrium of segment II :

$$H_{II/III} = H_{I/II}\left\{\frac{1 - B\tan\frac{\theta}{2}}{1 + B\tan\frac{\theta}{2}}\right\} - \left\{\frac{2F\frac{\tan\theta/2}{\theta} + 2rLA\tan\frac{\theta}{2}}{1 + B\tan\frac{\theta}{2}}\right\} \quad . \quad (4)$$

Similarly for limiting equilibrium of segment III :

$$H_{III/IV} = H_{II/III}\left\{\frac{1 - B\tan\frac{\theta}{2}}{1 + B\tan\frac{\theta}{2}}\right\} - \left\{\frac{2rLA\tan\frac{\theta}{2}}{1 + B\tan\frac{\theta}{2}}\right\} \quad . \quad . \quad (5)$$

Equation (5) can be used to calculate the thrust at the longitudinal joints of subsequent segments round the ring as far as the point of symmetry diametrically opposite the keying segment.

The average radial pressure on any given segment is readily determined from the average hoop thrust on its longitudinal joints.

Comparison of theory with observations

The results of the hoop-loading tests on ring 981 can be compared with the above analysis by using equation (5) and the values for the laboratory determination of the friction between concrete and clay.

The constants used are :—

$r = 56$ in., $L = 21$ in., $\theta = 36°$, $A = 5$ lb/sq. in, $B = 0{\cdot}34$ giving the following relations between the thrusts at segments 4 and 8 for the two limiting conditions when H_4 is the lesser and greater thrust respectively :—

$$H_4 = 0{\cdot}43 H_8 - 4{\cdot}6 \text{ tons}$$
$$H_4 = 2{\cdot}3\ H_8 + 10{\cdot}7 \text{ tons}$$

These relations are plotted on the graph of observed results, Fig. 16, and it will be seen that for increasing thrusts the agreement is good although the frictional loss is probably slightly underestimated. Most of the transverse joints had been roughly filled with mortar, and the discrepancy results almost certainly from friction at these joints, the effect of which was neglected entirely in the analysis.

During unloading the friction is considerably underestimated and it is probably caused by the concrete bonding to the clay under the higher pressures of the test.

In Fig. 21, theoretical distribution of hoop thrust is compared with the corresponding observed distribution of deformation and, although the deformation distribution is not perfectly symmetrical, the two diagrams show similar trends and suggest that thrusts and deformations are interrelated. To check this, the data for cycle 2 have been rearranged so that the theoretical thrust in each segment, calculated from the observed jack thrust, has been plotted against the corresponding radial movement as measured. In addition, the theoretical mean thrust has been plotted against the mean radial movement and all the graphs are shown in Fig. 22. It will be seen that after an initial curved portion arising from reloading, the graphs for individual segments, and also for the mean values, all become linear and have similar slopes. On this result it may be considered that the clay behaves elastically for the range of stresses applied, and that the mean thrust and deformation give a reasonable representation of the behaviour of the ring as a whole. It also confirms that the non-symmetrical radial deformation shown in Fig. 21 was caused by erratic extra friction in the lower part of the tunnel.

The analysis may also be used to estimate the hoop thrusts developed in a ring during the course of erection by the thrust of the shield ram on the key segment. For this purpose the angle of friction between the bitumen painted longitudinal sides of the concrete segments has been taken as 8°. This value has been adopted because it gives a rough agreement with the observed thrusts but it is also in reasonable agreement with the laboratory results. A 2° increase in this angle of friction would decrease the calculated thrusts by about 10%.

Using the values previously given, together with $\phi = 8°$, $\phi' = 22°$, $\alpha = \tan^{-1} \frac{1}{10}$, Table 5 compares the observed thrusts against those calculated from observations of the ram pressure gauge.

The keying segment (3, 5, or 9) which closed the ring is numbered on the standard system from the soffit segment as No. 1, whilst for convenience the joints are numbered from the key segment as No. 1, as was used in the analysis.

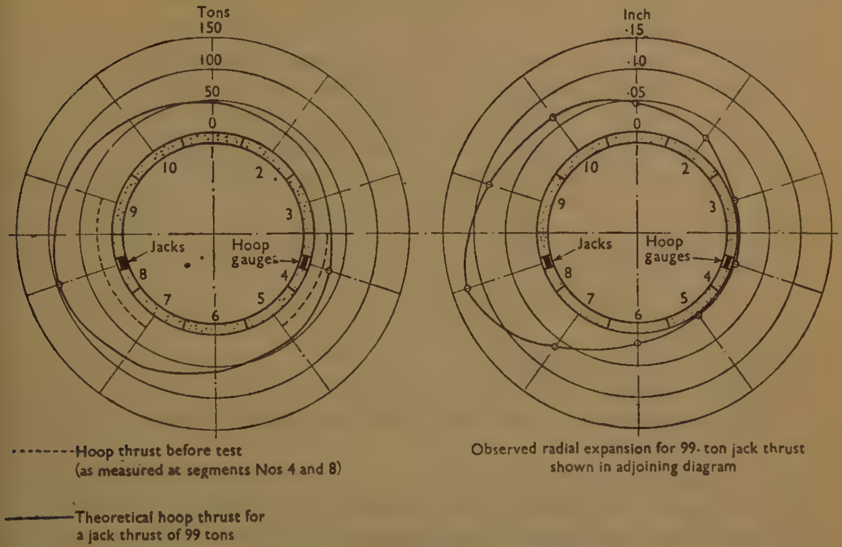

Tons
150
100
50
0
Inch
·15
·10
·05
0
10
2
3
9
Jacks
Hoop gauges
8
4
7
5
6
........Hoop thrust before test (as measured at segments Nos 4 and 8)
———Theoretical hoop thrust for a jack thrust of 99 tons
Observed radial expansion for 99-ton jack thrust shown in adjoining diagram

FIG. 21.—THEORETICAL HOOP THRUST AND OBSERVED DEFORMATION DURING A LOADING TEST ON RING NO. 981

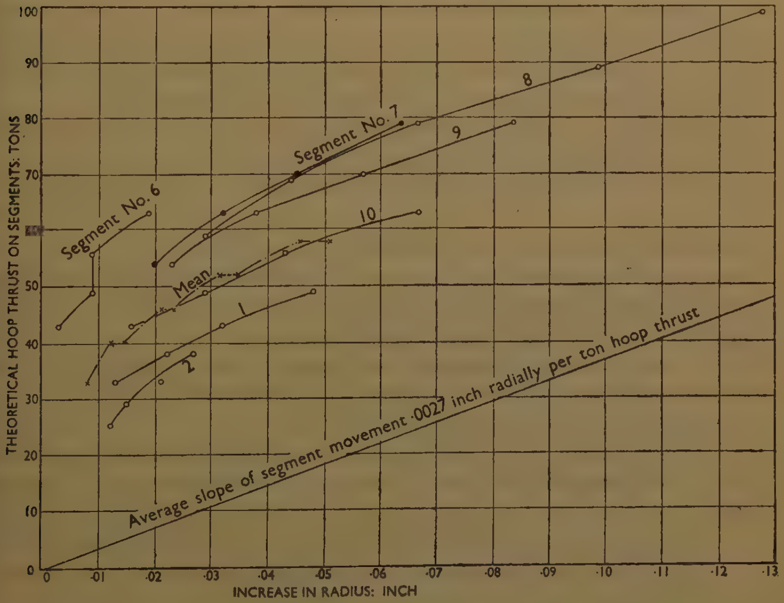


FIG. 22.—LOADING TEST ON RING NO. 981. RELATION BETWEEN THEORETICAL THRUST AND RADIAL MOVEMENT OF INDIVIDUAL SEGMENTS

TABLE 5

Ring		Key segment	Ram pressure: lb/sq. in.	Thrust at joint: tons (numbered from key segment)										
				I/II		II/III		III/IV		IV/V		V/VI		VI/VII
755	obs.	9	1,200		—		—		13		—		13	
	calc.			30		19		13		9		6		6
756	obs.	3	1,800		—		—		18		—		10	
	calc.			46		30		22		17		12		12
981	obs.	5	1,200		—		—		7		—		—	
	calc.			30		19		13		9		6		6
993	obs.	5	2,000		—		—		21		—		—	
	calc.			51		34		25		19		14		14

For segment IV of all four rings, the agreement between theory and observation is reasonable. If on erection, a ring expands symmetrically, the circumferential movements of segments V, VI, and VII diametrically opposite the key segment are probably insufficient to develop the full frictional resistance. Ring 755 was observed to expand symmetrically and this gives a reason for the high observed thrust at segment VI. On the other hand ring 756 expanded asymmetrically and segment VI was observed to move, so that the measured thrust is nearer the theoretical value. It is also of interest to note that when a ring is tightened, theory shows that up to 82% of the ram thrust is required to overcome friction.

Considering the number of indeterminate local variations and the limited number of practical observations, the analysis appears to give a reasonable estimate of the hoop thrusts immediately after erection.

Relation between the diameters of the shield and the lining

It is this predetermined relation which determines the amount by which the clay is recompressed on the erection of a ring, and the resulting ring pressure. It is discussed in detail on p. 405.

The following observations were made during the construction of the tunnel but, through limitations of time and suitable apparatus, they are insufficient to investigate the effect fully.

(1) When the shield was measured after completion of the tunnel, its average diameter was 0·2 in. larger than the average diameter of the tunnel lining. As far as could be determined, the shield appeared to be circular, although in the tunnel the vertical diameters of the lining were consistently about $\frac{1}{2}$ in. greater than the horizontal ones.

(2) While the key segment was being driven home, the horizontal diameter of a number of rings was observed to increase by

about 0·2 in. It was not possible to measure the vertical expansion, and it was not known how much of the 0·2 in. resulted from bedding down of the segments.

(3) The average observed radial pressure immediately after the erection of a ring was 2·1 tons/sq. ft. If the expansion of a ring during tightening follows the same relation as the hoop-loading test on ring 981, this pressure would be equivalent to a change in diameter of about 0·1 in.

OBSERVATIONS WITH THE TUNNEL FILLED WITH WATER

The testing programme, which began with the tunnel empty, was continued to investigate the effect of internal water pressure, and a number of additional instruments were developed to record any movement of the lining.

The vibrating-wire gauge for measuring changes in tunnel diameter, Fig. 23, bolts on to the wall of the tunnel and contains a vibrating-wire unit similar to that used in the load gauges. The 0·018-in.-dia vibrating wire is stretched between the base of the gauge and one end of a coil tension spring, the other end of the spring being connected by a 0·048-in.-dia. rustless wire to the tunnel wall diametrically opposite the gauge.

The tension in the wire across the tunnel is adjusted and kept almost constant, by a compression spring in the gauge head, whilst the initial tension in the vibrating wire is independently set by adjusting the tension spring. Any subsequent change in tunnel diameter alters the tension of the vibrating wire, and correspondingly, its frequency of vibration which is recorded and compared in the same way as the load gauges.

The gauge screws into a watertight outer casing and axial movement of the outer end of the tension spring is accommodated by a flexible metal bellows. Thrust on the end of the bellows, when the tunnel is charged with water, is eliminated by compressed air supplied through a 2-mm-bore P.V.C. plastic tube connecting the interior of the gauge to an air bell, which in turn is open to the water pressure in the tunnel. The air volume in the bell can be increased when necessary by supplying compressed air from the surface through another P.V.C. tube.

Before use the gauges were calibrated in a constant-temperature room so that movement at the remote end of the wire across the tunnel was directly related to the reference-gauge reading. With a maximum travel of $\frac{1}{4}$ in. the gauges were sensitive to 0·0005 in., although the average accuracy dropped to about ±0·001 in. for a complete test cycle.

The diameter gauges were installed near the centres of segments and, with one exception, on three diameters of each ring, these being the vertical (1–6) and the two nearest the horizontal (3–8 and 4–9). (See Table 1.) The wires across the tunnel were shielded from the flow of water by being

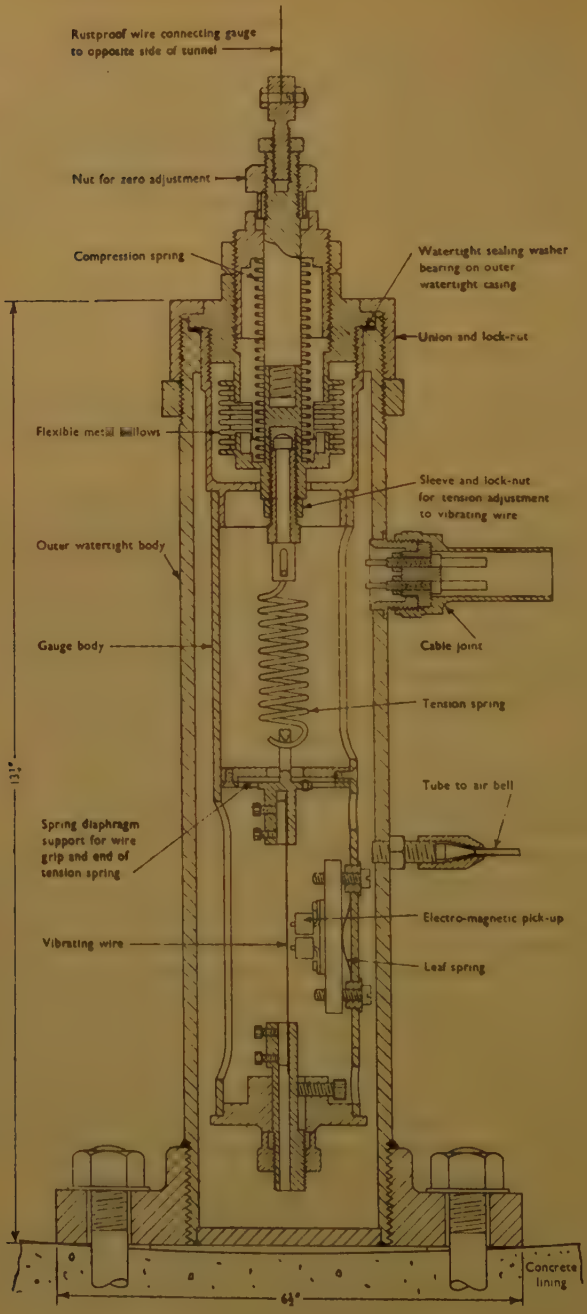

Rustproof wire connecting gauge to opposite side of tunnel
Nut for zero adjustment
Compression spring
Watertight sealing washer bearing on outer watertight casing
Union and lock-nut
Flexible metal bellows
Sleeve and lock-nut for tension adjustment to vibrating wire
Outer watertight body
Cable joint
Gauge body
Tension spring
13½"
Tube to air bell
Spring diaphragm support for wire grip and end of tension spring
Electro-magnetic pick-up
Vibrating wire
Leaf spring
Concrete lining
6¼"

FIG. 23.—VIBRATING-WIRE DIAMETER GAUGE

passed through 1-in.-dia. steel tubes; the general view of a test ring is shown in Fig. 24, facing p. 413.

Although the vibrating-wire units are unaffected by temperature changes, diameter-gauge readings are affected by temperature changes in the wire across the tunnel. In order to correct for this, tunnel temperatures were measured remotely by "Stantel" type K.B. thermistors. These were individually sealed into grease-packed brass capsules and installed in pairs at rings 352 and 981. Thermistors are thermally sensitive electrical resistances which with the simple ohm-meter in use, gave observations accurate to about $\frac{1}{2}$°F. After being in use for 18 months, the greatest change in calibration of any thermistor was found to be 0·6°F.

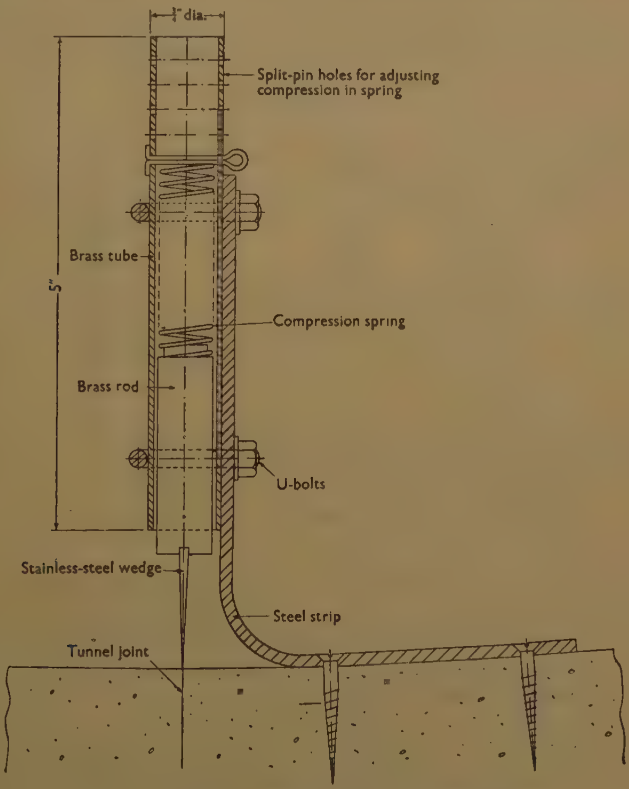
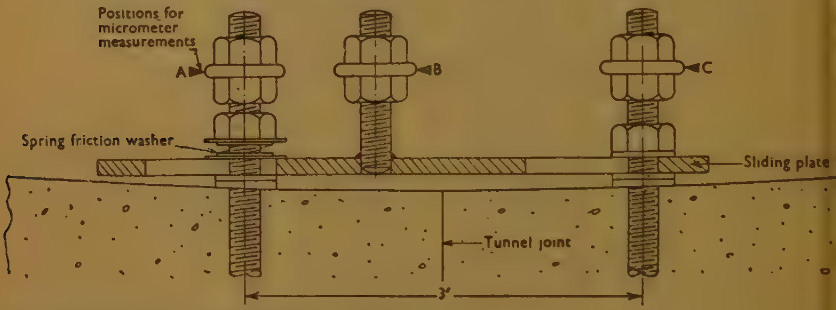


FIG. 25.—JOINT-MOVEMENT GAUGE: WEDGE TYPE

The experimental procedure for reading the diameter gauges was to take the zero readings when the tunnel was just flooded to the roof. Movement of the lining owing to this small water pressure is negligible and the rather inaccurate temperature corrections occasioned by taking zero readings when the tunnel was in free air are eliminated. Subsequent diameter-gauge observations during the testing cycle were corrected to the temperature when the zero readings were taken. No allowance has been made for temperature changes in the concrete lining or the clay. Observations of absolute change in diameter were continued by means of the micrometer measuring rod whenever the tunnel was empty.

As a check on the vibrating-wire diameter gauges, two different types of joint movement gauges were fitted across a number of longitudinal joints. These indicate any opening of the joint which may have occurred since their previous setting. One type, Fig. 25, utilizes a thin stainless-steel wedge of known taper which is pressed between the edges of the joint by a light spring. The leading edge of the wedge is only about 0·005 in. thick and the maximum opening of the joint is indicated by the measured penetration of the wedge.



Note: Measurement over AB records maximum movement of joint
Measurement over AC records final movement of joint

FIG. 26.—JOINT-MOVEMENT GAUGE: LINK PLATE TYPE

The other type, Fig. 26, consists of a brass link plate held in place by two brass studs fixed to the segments on either side of the joint. The link plate is moved by one of the studs if the joint opens and is retained in its new position by a friction washer on the other stud. Micrometer-screw measurement across the studs fixed to the segments gives the final movement, whilst measurement between one of the outside studs and a third stud fixed to the link plate itself gives the maximum opening of the joint.

Both joint gauges have an accuracy of about ±0·001 in., and approximately twenty-five of each type were installed, the majority being on the same rings as the vibrating-wire diameter gauges.

A number of short experimental cement surfaces applied to the inside of the tunnel lining also give a useful indication of any relative movement between the segments. Details of these are given in Table 2.

The positions of all the measuring instruments are shown in Fig. 1 and Table 1.

Damage during tunnelling made it necessary to repair the load gauges from four radial earth-pressure gauges, and during the absence of each gauge, pressure on the load plate was maintained by a spacing piece. After replacing a gauge the load was set as accurately as possible to its original value by adjusting the bolts retaining the cover plate (Fig. 3). In most cases, however, the adjusted load dropped immediately the tunnel was flooded. Subsequent changes in reading were consistent with those of undisturbed gauges, and the replaced gauges were then considered to have a constant zero error.

The electrical apparatus was waterproofed and connected by multicore cable, so that all the instruments could be read from the surface while the tunnel was in use.

During 1953 and 1954 the tunnel was subjected to three complete cycles of filling with water under pressure and emptying, the time record of water head and temperature being shown in Figs 27 and 29.

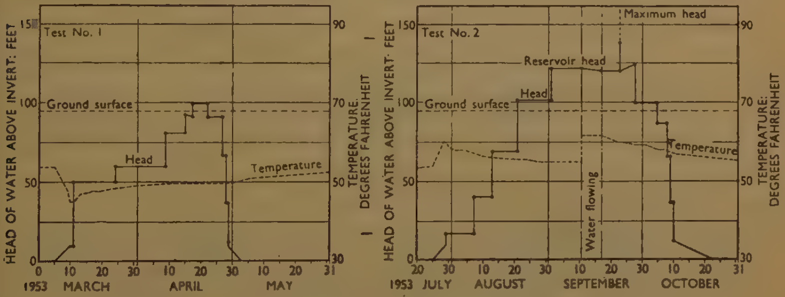


FIG. 27.—WATER HEAD AND TEMPERATURE DURING TEST FILLING NOS 1 AND 2

During the second cycle the head was increased to a maximum of 160 ft above invert but, owing to leakage past a temporary bulkhead in the unfinished surface conduit, this pressure could not be maintained. However, a head of 140 ft above invert was maintained sufficiently long for a set of readings to be obtained.

In the third cycle the tunnel was maintained at Queen Mary reservoir level for more than 7 months in order to investigate longer term changes from water pressure. No leakage of water from the tunnel has been detected.

During the periods when the tunnel was filled to Queen Mary reservoir level, water was flowing through at varying velocities up to 0·75 ft/sec.

Observations were made of water head and temperature, earth pressure and diameter changes and, after each cycle, the tunnel was thoroughly inspected and repairs made to the apparatus where necessary. During the time that a load gauge was out of action, the change in its reading was assumed to be equal to the average change in reading of all the remaining gauges in the ring.

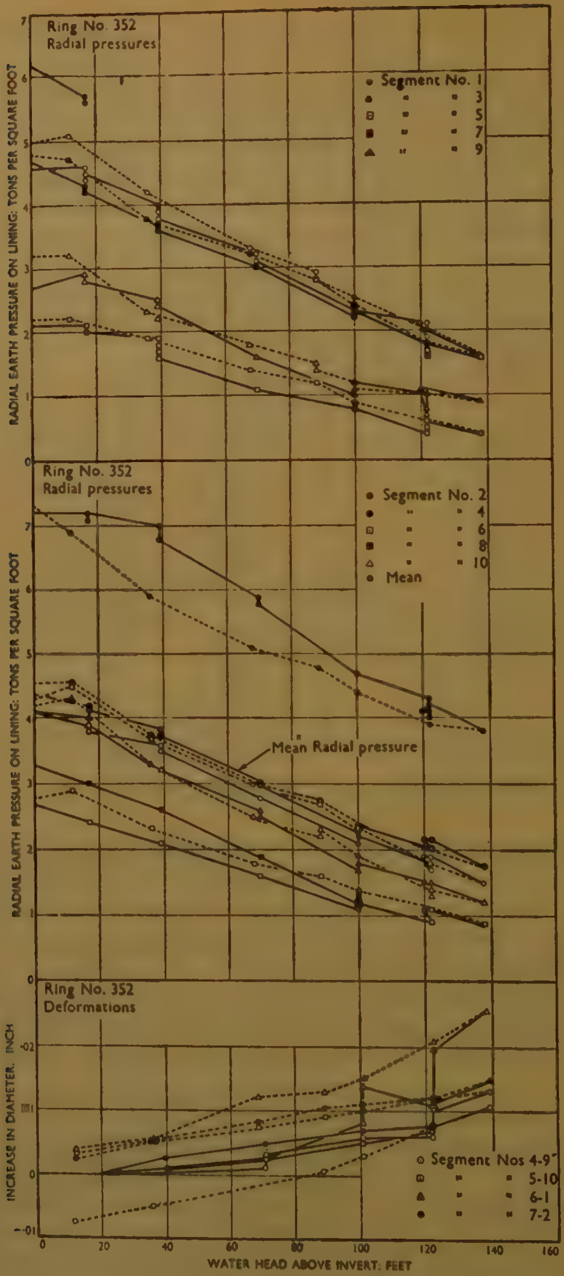


FIG. 28.—RESULTS FOR WATER-FILLING TEST NO. 2 (JULY–OCT. 1953). RING NO. 35

(Full lines denote filling; broken lines denote emptying)

Up to the end of the third test cycle, seven radial pressure gauges and eight of the load gauges used to measure hoop thrusts had developed leaks. The hoop-thrust gauges had not been designed for the tunnel and their waterproofing deteriorated during use, but on ring 756 a complete set of readings was obtained to the end of the second cycle.

The zero-load readings of all the radial pressure load gauges were checked at the end of the third test cycle, 2½ years after their original installation. This was accomplished without removing the load gauges by slackening the bolts retaining the cover plates (Fig. 3). As soon as the zero reading had been obtained the gauge was reloaded to its original reading. The entire operation occupied less than one minute and had a negligible effect on the continuity of the pressure observations. The average zero shift in 2½ years was equivalent to a gauge reading low by 0·25 ton/sq. ft of radial pressure, the worst change being 0·5 ton/sq. ft. Since the original zero readings were taken without detailed checks against the stable tuning fork (see p. 412) it is impossible to estimate how much of the change arose from reference gauge instability. The results, however, confirm the reliability of vibrating-wire gauges for long-term observations.

A number of vibrating-wire diameter gauges failed during the first two testing cycles owing to minor faults in details which were not appreciated during design, but after slight modifications and repair no further failures occurred.

Figs 27 and 29 show the changes in water head and temperature during the three test fillings, Figs 28 and 29 show typical observations of ring pressure and deformation, and Figs 30 and 31 show changes in average radial pressure per ring during the three test fillings.

The results showed that the net earth pressure on the tunnel lining decreased with increasing internal water pressure and increased again as the water pressure was reduced. Most of the change in lining pressure occurred very rapidly during the time required to alter the water pressure, but small changes continued for some time later. In tests 1 and 2 the tunnel was filled at a slow rate such that lining pressures appeared to have become steady at each applied water pressure, whilst in test 3 it was filled more rapidly. The tunnel was emptied at approximately the same rate in each test.

For the first filling the rate of decrease of lining pressure was rather irregular, but while emptying the tunnel, and during the second and third tests, an almost linear relation was obtained between earth pressure on the lining and the internal water pressure. Observations of changes in both radial earth pressure and hoop thrust were generally consistent, and showed that the change in radial earth pressure on the lining was about two-thirds of the change in internal water pressure. This ratio did not vary significantly with the rate of filling the tunnel, but was slightly higher for emptying than filling. The results are detailed in Table 6.

The original large variations in radial pressure round the rings were not

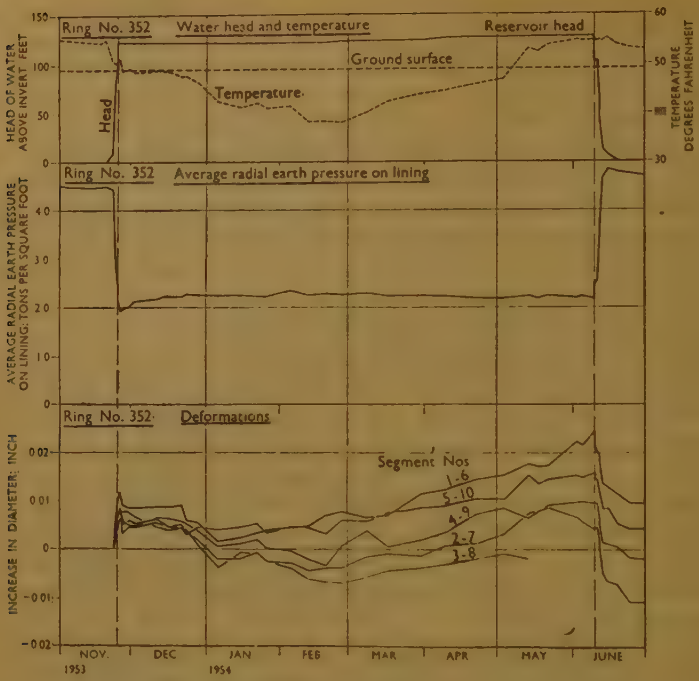


Fig. 29.—Results for water-filling test no. 3 (nov. 1953–june 1954). Ring no. 352

eliminated when the tunnel was filled with water but some redistributio occurred during each filling, particularly the first. There was also tendency for a small overall increase in radial pressure by the end of eac test cycle which can be seen in the graph of average radial pressures Figs 30 and 31.

Since the net radial earth pressure on the lining decreased by onl about two-thirds of the applied internal water pressure, none of the tes rings ever became completely unstressed, even though the internal wate pressure was greater than the average radial pressure on these rings whe the tunnel was empty. Because of this changes in diameter were everywhere small and appeared in general to show a linear increase with increasing internal water pressure. These small changes are a function the relative elasticities of the clay and concrete ring, and at Queen Mar reservoir head the increases in diameter were generally between 0·01 an 0·02 in.

All the movement gauges fitted across the longitudinal segment joint consistently showed that no measurable movement had occurred. Th result is in agreement with pressure measurements, and is confirmed by

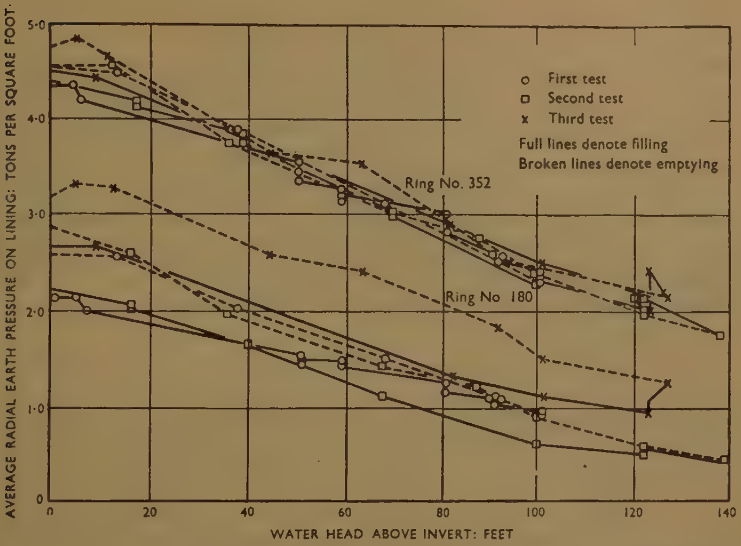


FIG. 30.—AVERAGE RADIAL EARTH PRESSURE ON LINING DURING WATER-FILLING TESTS. RING NOS 180 AND 352

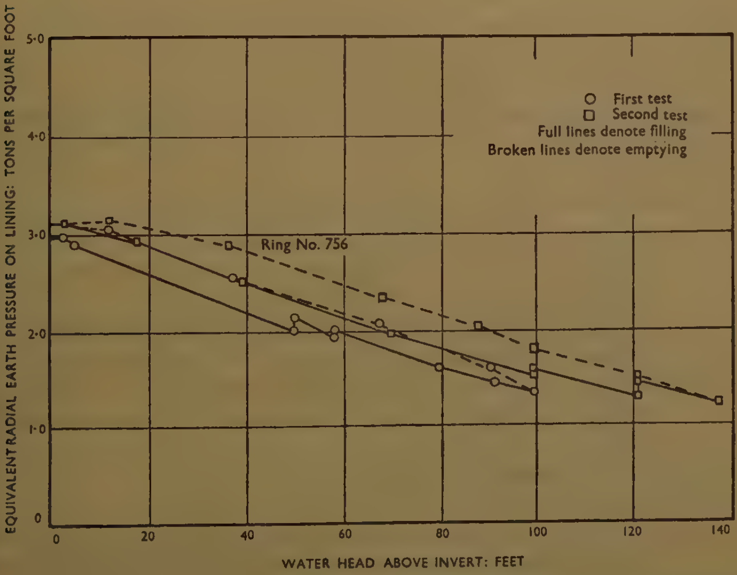


FIG. 31.—RADIAL EARTH PRESSURE ON LINING COMPUTED FROM AVERAGE HOOP THRUST DURING WATER-FILLING TESTS. RING NO. 756

visual inspection of the tunnel and its trial lengths of cement rendering, all of which were in perfect condition.

The longer-term effects of internal water pressure were investigated in test 3, and Fig. 29 shows the results for ring 352 which are typical of the behaviour of the tunnel. The earth pressure on the lining increased slightly for about 1 month after the internal water pressure was applied, and thereafter remained sensibly constant during the subsequent 6 months. Immediately after emptying the tunnel, the average radial pressure on the lining was rather greater than at the start of the test, but pressures decreased slightly so that by the end of the following 2 months, a steady pressure had been attained of approximately the same value as at the start of the test.

The tunnel diameters, after their initial increase with internal water pressure, followed the variation in tunnel water temperature, as shown in Fig. 29. However, superimposed on the temperature effect was a second variation in which the lining was changing shape, the vertical diameter increasing while the horizontal diameter decreased. This change in shape is opposite to that recorded when the tunnel was empty and is best seen in the records of diameter changes measured by the micrometer rod, Figs 12 to 14. The changes in shape are very small, the greatest change in diameter being about 0·02 in., and 6 months after the end of test 3 the tunnel had reverted to the stable shape attained before any of the filling tests were commenced.

Before the tunnel is charged with water, the loading on the lining consists of a radial pressure and circumferential shear, both of which vary round the ring. Internal water pressure superimposes on this system a uniform opposing radial pressure. A rigorous solution of the theoretical relation between internal water pressure, and the lining pressures and deformations is highly complex but it is of value to check if simple theory gives a reasonable result.

The assumptions are :—

(1) The internal water pressure is directly transmitted to the clay as if there were an impermeable flexible membrane at its inner surface.

(2) Both the concrete lining and the clay behave as homogeneous elastic materials.

(3) During the time occupied in filling or emptying the tunnel, any effect due to the swelling or consolidation of the clay is negligible.

(4) The circumferential shear is ignored.

(5) The radial pressure and deformation affecting the lining are uniform. The uniform values are equal to the average of the observed values for each ring. This assumption is to some extent justified by the results of the hoop-loading test carried out on ring 981 when the tunnel was empty.

p_o denotes average radial pressure on lining with the tunnel empty.
p_w ,, internal water pressure.
p_e ,, average net radial pressure on lining at a given internal water pressure p_w.
r_o ,, outside radius of tunnel lining.
$\bar{r}$,, mean radius of lining.
t ,, thickness of tunnel lining.
u ,, radial movement of outside of lining at a given internal water pressure p_w.
E_e ,, apparent modulus of elasticity of the clay for the conditions of the analysis.
E_c ,, modulus of elasticity of the concrete lining.
μ ,, Poisson's ratio for clay.

Provided that the hoop thrust in the lining is greater than zero, the radial movements of the outside of the lining and inner surface of the clay are equal. In terms of the pressures on the lining and on the clay, they are respectively :

$$u = \frac{p_o - p_e}{E_c} \, \frac{\bar{r}\, r_o}{t} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$$u = \frac{p_e + p_w - p_o}{E_e} (1 + \mu) r_o \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

from which :

$$p_o - p_e = \frac{p_w}{\left(1 + \dfrac{\bar{r}}{(1 + \mu)t} \, \dfrac{E_e}{E_c}\right)} \quad . \quad . \quad . \quad . \quad (3)$$

Using this theoretical analysis, the moduli of elasticity of the concrete lining and the clay have been calculated from the observed changes in average radial earth pressure, and average change in diameter, for both the loading and unloading part of each cycle.

Poisson's ratio was taken as 0·5 on the assumption that the clay is incompressible.

There was no significant difference between the moduli of elasticity for filling and emptying and, apart from some erratic values during the first test filling, the values were fairly consistent.

In Table 6 are shown for each ring the average values for the three filling tests.

The average modulus of elasticity of the concrete, determined by compression tests on six individual concrete segments 2 years after casting, was 5·7 × 10^6 lb/sq. in. Tests to investigate the effect of compression of the joints indicated that for well fitting joints the overall modulus of the ring could be about 80% of that for a single segment.

Considering the relatively low accuracy of the calculated moduli of elasticity owing to the small tunnel deformations, and also that ring 756

TABLE 6

Ring	$\frac{p_o - p_e}{p_w}$ *		* d: in.	Modulus of elasticity: lb/sq. in.		
	Filling	Emptying		No. of determinations	Concrete lining	Clay
179	0·64	0·75	0·014	3	$2 \cdot 5 \times 10^6$	$2 \cdot 5 \times 10^5$
180	0·57	0·68	0·010	4	$2 \cdot 8 \times 10^6$	$3 \cdot 0 \times 10^5$
352	0·75	0·85	0·010	6	$4 \cdot 1 \times 10^6$	$1 \cdot 5 \times 10^5$
756	0·61	0·63	0·016	4	$1 \cdot 4 \times 10^6$	$2 \cdot 5 \times 10^5$

* $\frac{p_o - p_e}{p_w} = \frac{\text{change in net earth pressure on lining}}{\text{change in internal water pressure}}$

d = increase in diameter at normal working head (120 ft above axis).

included two pairs of circumferential load gauges, the values for the concrete lining are reasonable.

This leads to the conclusion that the modulus of elasticity of the clay for the particular conditions investigated is of the order of $2 \cdot 5 \times 10^5$ lb/sq. in. The average value obtained from seventeen triaxial compression tests in the laboratory on samples $1\frac{1}{2}$ in. dia. and $3\frac{1}{4}$ in. long, prepared with the minimum of disturbance, was $0 \cdot 37 \times 10^5$ lb/sq. in. There is considerable difference here, but the correlation between various methods for the determination of the modulus of elasticity of soils has not yet been fully investigated. The value of $2 \cdot 5 \times 10^5$ lb/sq. in. obtained from tests on the tunnel is reasonable when compared with the modulus of elasticity of rocks of approximately similar strength. It may, therefore, be concluded that this analysis gives reasonable results providing a suitable value for the modulus of elasticity of the clay is used.

DESIGN CONSIDERATIONS

Friction during erection

A final ram pressure on the key segment of 3,000 lb/sq. in., producing a ram thrust of 51 tons was not very exceptional. Neglecting friction, this should produce a hoop thrust of 255 tons. The maximum hoop thrust measured immediately after jacking was only 21 tons and the average thrust immediately after jacking was only 17 tons. Also, a direct jacking force of 99 tons, as described on p. 427, produced only 35 tons hoop thrust at an angular distance of 144°. It was, therefore, obvious that very high frictional forces were developed during erection.

It was primarily to find means of reducing this friction that the laboratory experiments described in the section on the mechanics of erecting the lining were undertaken. It was established that the three main sources of friction were the sliding concrete surfaces of the key segment, the transverse movement of the segments adjacent to the key against the corresponding segments in the previous ring, which provide the reaction to the ram thrust, and the movement of the segments over the surface of the clay. In the first two cases the friction is caused by concrete surfaces under very high pressures of the order of 2,000 lb/sq. in. In the last case, the friction is between concrete and clay with average bearing pressures only about 50 lb/sq. in.

Search for suitable lubricants was concentrated on two main groups of materials, bituminous preparations, and more normal lubricants of the grease type. It was found that friction between concrete and concrete under high pressure was best reduced by a stiff thick lubricant which had sufficient thickness and strength to keep the two concrete surfaces apart and provide a shearing layer. Provided the rate of movement is not too high, the friction is then dependent on the shearing strength of the lubricant. Many bitumen preparations were tried but the most suitable from all points of view was found to be a rubber latex/bitumen emulsion in the form of a thin paste, mixed with an equal weight of cement and 0·6 parts of water. This paste could be applied to the concrete surfaces by brush and produced a tough, elastic film with an average thickness of about 0·017 in. when dry. The final thickness of the film under load was about 0·01 in. Experiments in the loading frame (Fig. 19) gave angles of friction of only about $1\frac{1}{2}°$. The creep characteristics of this material were also investigated and found to be much better than plain bitumen. The mixture would form an excellent jointing material and would also be beneficial in securing uniform bearing on the cross-joints.

Friction between the segments and the clay takes place under much lighter loads and other lubricants are more suitable. Water effects a considerable reduction in friction but only when present in sufficient quantity. The results of these tests are given on p. 430 and for water $s = 3{\cdot}5 + 0{\cdot}13n$ lb/sq. in. It would not be possible under practical tunnel driving conditions to maintain a sufficient quantity of water on the clay face. Various kinds of grease and other materials were tried, but easily the most suitable was found to be ordinary soft soap. This material reduces the friction between concrete and clay under loads similar to those on the tunnel lining, to an almost negligible amount, i.e., $s = 0{\cdot}04n$. There are no practical objections to the use of soft soap which can easily be cleaned from the hands and there is no doubt that the use of suitable lubricants wherever possible would be advantageous with this type of lining.

A certain minimum value of the friction on the wedge surfaces must be maintained, however, or the wedge or wedges will be forced out when the ram is withdrawn. Neglecting certain secondary forces, the angle of friction should not be less than the half angle of the wedge. The rubber/bitumen mixture gave values much less than this and it would be unwise to use it on the wedge surfaces without further trial.

Improvements in design

It seems highly desirable that movement of the segments during tightening of the lining should be avoided, if possible. In the Don-Seg design movement is necessary so that the last segment can be inserted radially. Movement of the segments during tightening could be avoided if only a single tapered key segment was provided and the remainder of the longitudinal joints were parallel to the centre-line. The key could then be inserted longitudinally from the leading end without serious inconvenience. In such a ring all the segments could be placed direct into their final position and there would be no movement of any segment during closure except the very small amount of radial movement owing to a slight increase in diameter.

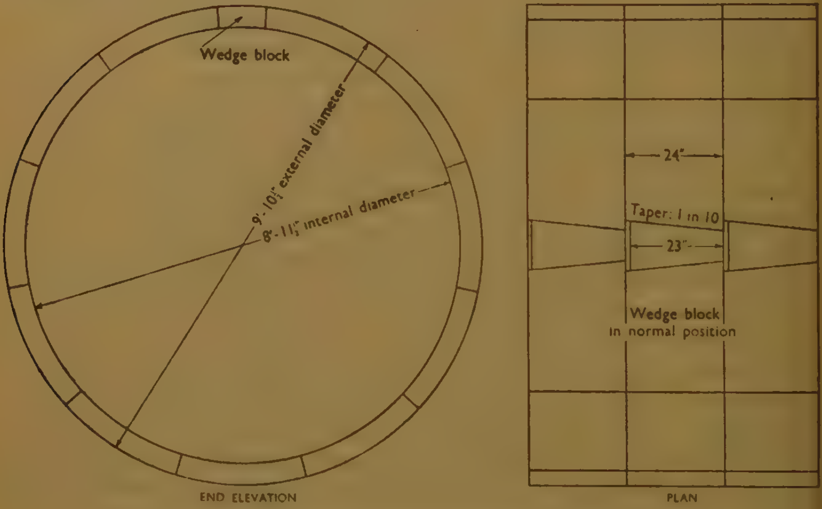


FIG. 32.—WEDGE-BLOCK PRECAST-CONCRETE TUNNEL LINING

It is difficult to see how the segments in such a lining could become displaced during tightening and it might confidently be expected to be better fitting with more uniform bearing between segments, have lower maximum stresses, and a constant prestress in the ring.

It is also very desirable that a small latitude in the circumferential length of each ring should be available. This can be obtained by making the key segment shorter in length than the normal type and designed so that the small end, when in the normal position, is at the rear end of the ring. There would then be a small gap at the end of each key segment which could afterwards be filled up by hand. The required latitude in jacking the ring would be obtained by over- or under-driving the key segment relative to its normal position. A lining of this type is shown in Fig. 32.

The Board intend to construct a further length of tunnel in the near future to these improved designs and using lubrication between the segments and the clay.

Steel linings

The two chief reasons advanced for the use of steel linings are that a steel lining is watertight (which the clay may not be) and that it is necessary to carry the water pressure. The steel plate can be assumed to be watertight but it is difficult to ensure that all the joints will be watertight. This can only be guaranteed by testing every joint, which is a very difficult operation with large-diameter thin pipes. The fact that a steel pipe is not necessary to carry a reasonable degree of pressure has been demonstrated by the investigations described in the Paper.

The main disadvantage of a steel-lined tunnel is its cost, which is almost double that of an unlined tunnel for the same finished diameter. Besides providing and placing the actual steel pipe, the outer lining must be of larger diameter, the space between the pipe and outer lining must be grouted solid, and the pipe must be protected on the inside. It is particularly difficult to ensure that external grouting will be perfectly sound and satisfactory.

There is also risk of collapse of the steel lining owing to external pressure when the tunnel is empty. It was shown in the Ashford Common Tunnel that water passes through the clay into the tunnel. The quantity is infinitesimal from an engineering point of view, but if the tunnel had an impermeable steel lining there would be a danger of building up a large hydrostatic pressure on the outside of the lining, sufficient to cause the steel lining to collapse. The danger would be increased by any areas of bad grouting and the prevention of collapse would probably depend on the adhesion of the grout to the steel.

A satisfactory drainage system to eliminate external hydrostatic pressure would be extremely difficult to construct in London Clay. Drains at conventional spacings in such clay would tend to be purely local in their effect and the only really satisfactory drainage system would be a continuous cover to the pipe of porous grout.

It is also by no means certain that even if a steel lining was provided it would necessarily carry the tension resulting from the water pressure. If, for instance, the external grouting was really well done, the steel lining ought to be in compression before filling with water and the outward pressure of the grout carried by the clay through the outer lining. Subsequent internal water pressure would reduce the compression in the steel but increase the outward pressure on the clay. The final result might well be a steel lining carrying very little stress and a higher stress in the clay than with an unlined tunnel.

ECONOMIC CONSIDERATIONS OF TUNNELS AND SURFACE CONDUITS

The principal advantages of tunnels as opposed to surface conduits are as follows :—

(1) A tunnel can be driven in what is sensibly a straight line, giving the shortest route and lowest friction loss.
(2) Surface difficulties and traffic dislocation are avoided.
(3) There is a saving of land.
(4) For all but the smallest sizes, the cost of tunnels without a steel lining is much less than that of surface conduits. Moreover, the rate of increase of cost with diameter is greater for surface conduits than for tunnels.
(5) A tunnel can usually be placed below the hydraulic-gradient line so that the water can be allowed to flow by gravity and pumped, if necessary, at the terminal point. This allows more flexibility in the placing of pumping stations and reduces the head on the tunnel.

These are important advantages. Fig. 33 shows the estimated overall inclusive costs per linear yard of surface conduits and tunnels without a steel lining in the Thames Valley. The total cost will in many cases be further reduced in the case of tunnels because of their shorter length and the avoidance of special difficulties.

The economical diameter of a tunnel is likely to be greater than that of a corresponding surface conduit when pumping charges are taken into account. Not only does the cost of tunnels increase more slowly with diameter than surface conduits but friction falls rapidly with increasing diameter. This further reduces both the capital cost of the pumping plant and the running costs.

These advantages may be illustrated by the following example. It is admittedly a very favourable example but it represents an actual practical case.

The Metropolitan Water Board have powers to construct two large reservoirs in the Thames Valley. When these powers were obtained in 1946 it was intended to connect the outlets of these reservoirs to that of an existing reservoir and with filtration works in the same area, all by means of surface conduits. To empty the reservoirs and to overcome the friction head, it would have been necessary to construct a large pumping station at each reservoir, with a total installed capacity of the order of 12,000 w.h.p. The total length of conduit would have been about 10¼ miles, all subject to a high head because of the high friction from a relatively small diameter conduit.

It has now been decided to replace these conduits by a deep-tunnel system. The tunnels will be driven in approximately straight lines from station to station and the total length will be only 8½ miles against 10¼ miles for surface conduits. The tunnel will also be made considerably larger in diameter than would the surface conduits, reducing very considerably the friction head. The tunnel will be well below the hydraulic-gradient line all the way so that the water will flow by gravity to the end of the tunnel. Here the water can be pumped by existing machinery which will lift the water to the filtration works. None of the three pumping stations at the reservoirs will be necessary since all the pumping will be performed at the existing station. The total pumping head in the tunnel scheme will be only 40 ft. In the surface conduit scheme, the pumping head would have been about 100 ft.

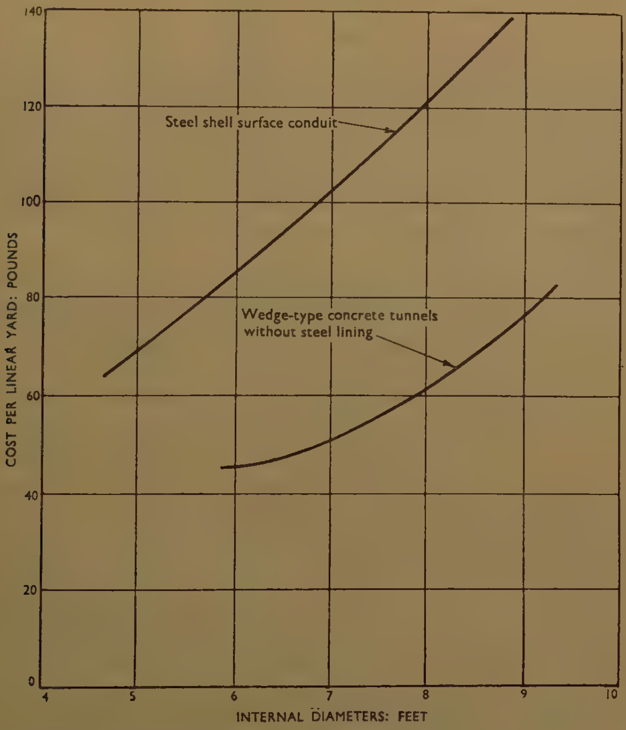


FIG. 33.—RELATIVE COSTS OF SURFACE CONDUITS AND TUNNELS

About 110 acres of land would have been required for the surface conduits and practically none for the tunnel scheme. The total cost of the tunnel scheme is about £1$\frac{3}{4}$m. less than the surface scheme.

An example which illustrates the saving in cost between an unlined tunnel and a lined tunnel, in circumstances not particularly favourable to the former, is the River Thames–Lee Valley tunnel referred to on p. 401. The original proposal for present requirements was for a 75-in.-dia. steel lined tunnel to carry 70 m.g.d., which required a friction head of about 115 ft.

It has now been decided to omit the steel lining but, to meet certain imposed conditions, the diameter has been increased to 102 in., which has reduced the friction head to 23 ft. The tunnel has now become a gravity tunnel and the pumping will be performed by submersible pumps in the terminal shaft. The saving in capital cost compared with the lined tunnel is about one million pounds.

CONCLUSIONS

The general conclusion is that, for reasonable pressures, a steel inner lining is not necessary with this type of tunnel in London Clay. The permissible pressure and the amount of prestress required in the tunnel rings must be determined by the circumstances of each particular case.

The individual conclusions for the particular case of the Ashford Common Tunnel can be summarized as follows :—

(1) The London Clay was of such a high strength that the earth pressures on the tunnel lining depended almost entirely on conditions during the erection of the individual rings.

(2) The important conditions during erection were :—
  (*a*) the relative sizes of the shield and the lining ;
  (*b*) friction on the sliding surfaces ;
  (*c*) badly fitting joints and misalignment of segments caused by erratic movement of segments during erection.

(3) Owing to friction, the pressure distribution round a ring immediately after erection was non-uniform and often irregular.

(4) Once built, the pressures on a ring were affected by the erection of the next few rings, and thence, little significant change was observed during the following year. The distribution of pressure remained irregular and the average radial pressure per ring was generally between 2 and 3 tons/sq. ft.

(5) As built, the lining was slightly elliptical with the vertical diameters about half an inch greater than the horizontal. Neglecting changes which occurred immediately after construction, the lining deformed at a decreasing rate for about one year when the vertical diameter had decreased by about 0·06 in., and the horizontal diameters had increased by about 0·02 in.

(6) While the tunnel was filled with water, earth pressures on the lining decreased as the water pressure was increased, and similarly increased again as the water pressure was reduced. The change in earth pressure on the lining was generally about two-thirds of the change in water pressure.

(7) The water pressure applied was insufficient to unstress any of the test rings, and diameter changes owing to water pressure were small. Under the normal working head of 120 ft at the axis of the tunnel the increase of diameter was between 0·01 and 0·02 in., and there was no measurable relative movement between adjacent segments.

(8) Under water pressure there was no measurable leakage through the London Clay.

ACKNOWLEDGEMENTS

The Ashford Common Tunnel was designed and constructed by the Metropolitan Water Board and thanks are due to Mr H. F. Cronin, C.B.E., M.C., B.Sc.(Eng.), M.I.C.E., Past President, Chief Engineer to the Board, for permission to present the Paper.

One of the Authors (Mr Ward) took part in this investigation as part of the research programme of the Building Research Board of the Department of Scientific and Industrial Research and has collaborated in this Paper by permission of the Director of Building Research.

Acknowledgements are made also to the following :—

The tunnel contractors, Kinnear, Moodie & Co. Ltd., who took a great interest in the experiments and were very helpful.

Finally, to many of the Authors' colleagues for much stimulating advice and discussion.

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The Paper, which was received on the 26th November, 1954, is accompanied by five photographs and twenty-one sheets of drawings from which the half-tone page plates, and the Figures in the text have been prepared.

Discussion

Mr T. N. C. Bulman said that the Paper was full of information, but he could touch on only a few points.

On p. 404 a depth of 59 ft was quoted. Accepting the hypothesis that earth pressure was double water pressure, he suggested to the Authors that it should be 46 ft. That was a small point, because later on the next page it was stated that a depth of 90 ft had been adopted, which suggested to him a factor of safety of 2.

In Table 3 on p. 408 it was stated that the density was 130 lb/cu. ft, which, at a depth of 90 ft, gave theoretically an earth pressure of 5·2 tons/sq. ft, which was much the same as one of the Authors had stated in introducing the Paper. As had been pointed out, Figs 7, 8, 9, 10, and 11 showed that the measured pressure was 3 tons/sq. ft. That reduced the apparent factor of safety to 1·2. A factor of safety was a ratio, and 1·2 : 1 did not leave a lot over for safety.

On p. 412, where the gauges were discussed, it was stated that the reference gauge, which he understood was a vibrating-wire gauge similar to the in-situ gauges, had proved so unreliable that it had had to be very frequently calibrated against a tuning fork. How did the Authors know that all the in-situ gauges had not been equally unreliable ? He would be very worried about that. In passing, could the Authors say how they had calibrated those vibrating-wire gauges against actual pressure ? Had they done it in the testing machine and the laboratory ?

On p. 427 and following pages the hoop test was discussed, and it was stated that 99 tons had been inserted in the circumference on one side of the tunnel and 37 tons had reached the other side, 62 tons presumably being lost, entirely owing—in the opinion of the Authors—to friction between clay and concrete. If that was correct, why was it that the friction did not work both ways and, in taking off those 99 tons, how was it that 37 tons got away and did not remain ? Again thinking of 5·2 tons/sq. ft and applying the friction formula which the Authors gave on p. 430 as $s = 5 + 0{\cdot}34n$ lb/sq. in., he arrived at the friction figure of 53 tons instead of the 62 tons that had been lost ; if he reduced that in the ratio of 3 : 5·2, there were 31 tons left. On that basis, again, he was not prepared wholly to agree with the Authors' contention that all the loss arose from friction. He thought that a great deal of it could be accounted for by the energy lost owing to deformation. There were also possibilities of friction between adjacent rings, and compression of the concrete which was, he supposed, the energy of deformation as well.

On p. 450 there was reference to a new design. There were gaps in his information, because he had to admit that there were parts of the Paper which he had not read thoroughly but, so far as he could see, nothing had been said to suggest that the tunnel as built on the principle described had proved to be in any way defective. It was suggested that an alteration be made and that, instead of having ten similar segments to form a ring, there were going to be approximately eight similar ordinary segments, two Tee-segments, one on each side different from one another, and a key. In practice that would involve more handling and more costs of manufacture. He suggested to the Authors, that, in applying the full hoop thrust necessary at the ring at one key only and not at five keys (as he understood it had been applied in the existing tunnel), that single point of application might lead to two difficulties in practice. One was that the key would tend to force its way out of the circle and into the clay, and the other was that possibly (he had not had time to check it mathematically) there would be a tendency for the stressing in the vicinity of the key—i.e., the compressive stresses in the concrete—to reach a point where the concrete would spall away. Very good concrete might stand up to it.

On p. 449, the Authors suggested the use of soft soap and said that it could easily be cleaned from the hands. The concrete segments weighed about 3 cwt and he thought it very undesirable that men handling them should have soapy hands, since that might lead to accidents; he did not agree that hands could easily be cleaned in a tunnel.

On p. 406 the Authors remarked in passing that the standards of accuracy imposed by engineers when tunnels were being built could be reduced. It was said that corrections of line and level needed to be made much less frequently in water tunnels than in railway tunnels. He could not agree as a practical engineer that the standards should be relaxed on that sort of work.

Finally, on pp. 405, 436, and 437 the Authors had stated that they were dissatisfied with existing shields. He was not at all clear what the dissatisfaction was, but he could make a guess; he thought that the Authors were concerned with the deformation of the shield whilst in use, which, on their own measurements, was about ½ in.; in a shield of 9 ft 5 in. dia. that was not at all excessive, and he doubted whether it could ever be improved upon. He did not think that a ½-in. variation in dia. imposed any difficulties.

Mr J. H. Seagrave added his thanks to the Authors for their very interesting and informative Paper.

The first thing that had struck him was the statement on p. 400 that "tunnels were generally considered to be far more expensive than surface pipes." The Metropolitan Water Board had now been driven underground, like one of the other big public undertakings, and when the terrific cost of traffic delays to the public was considered (and, after all, it was the public who paid for the Board's mains), he felt that the cost of the tunnel would be easily comparable with the cost of a trench, say, down Cheapside or Holborn for a water main of considerable size.

The Water Board's project of a main connecting the Thames to the Lee had fortunately been capable of construction in clay. That had given a marvellous chance to try out the Don-Seg lining, and he thought that the originator of that lining ought to be congratulated on the fact that the creation of his brain had stood up so wonderfully well to all the tests that had been put upon it.

Those tests were of great interest. The changes in diameter that had been observed when the tunnel was filled with water had been very small, the diameter of the tunnel apparently having increased by 0·01 in. Of course, the shield had not been constructed to an exact circle to 0·01 in. all round ; nor did he think that the makers of the segments would claim that they could build a circular tunnel exactly to within 0·005 in. of a true circle ; thus, a change in diameter of 0·01 in. was very little indeed. A usual feature in tunnels was that they always built big on one diameter and a little bit small on another, so that really a tunnel was obtained which was a little bit oval. Those Don-Seg tunnels had been extremely tiny in the oval diameter, which he thought was a wonderful thing.

The tunnels were not grouted, so that there was no constructional adhesion between the lining and the clay. With a concrete-lined tunnel, with concrete segments or with cast-iron segments, if the space between the segments and the clay was grouted, there was an absolute constructional adhesive connexion. That was not so in the Don-Seg tunnels. The thrust of the shield was felt many rings back—unlike what happened in the case of the ordinary shield where the grouting was spread right up to the shield—so, since the segments were wedge-shaped, there was sure to be a little bit more movement among the segments. Since about 0·01 in. variation in diameter was being considered and the segments would only fit just where they touched the clay to 0·01 in. (which was very little), the segments were really only settling into the very little space that there was.

The tunnel had not been truly circular to start with. In Figs 12, 13, and 14 the tunnel was already taller by about 0·3 in. and narrower by 0·3 in., but that was nothing to worry about. One then got a movement of 0·02 in. in diameter, and since there were ten joints to a circle, that was only 0·003 in. per joint. The bitumen painting was 0·017 in. thick ; that was 0·034 in. per joint. If there was a movement of only 0·003 in. that was only one-tenth of the thickness of the bitumen paint. Thus, the tunnel, when the water pressure was put in, was only just breathing in the bitumen jointing.

There was a note on p. 451 that an infinitesimal amount of water had passed through the clay into the tunnel. He thought it was not a question of passing through the clay into the tunnel. The clay was always wanting to consolidate and settle down, and it could only do so by getting rid of its water content (which was 25% of its weight in London Clay), so that if there was any cavity left in grouting where one set of grouting was joined up to the next in iron tunnels, the clay would seize the opportunity of squeezing a little of its water content out into that cavity. That generally happened after about 6 months. In a railway tunnel slight dripping might occur in about 6 months, but that was of no consequence at all; the draught of the trains evaporated the drips, and nothing further would be seen. But if dripping developed in a tunnel for cables and other things, there was soon quite a mess—one drip every second was more than a gallon a day! But when the grouting plug, which had perhaps been leaking was removed, and about half a cupful of water was taken out, that ended it. It was just water that had been squeezed out of the clay, and it got away wherever it could. That was a feature of every tunnel in clay that he had ever met.

The clay pressure did build up in time. A hole could be dug in clay and a lining put in and left for a while; but after a time the pressure came on. It was found in timber headings. If a timbered heading was to be used for a little while, the timber did not show any change, but if it were there for 6 months the strains began to be visible on the timber.

He thought it was wrong to think about the steel lining being prestressed by grouting in a tunnel lined with steel, as described on p. 451. The purpose of the grouting was really only to fill the annular space between the steel lining and the wall of the tunnel. That space could only be completely filled if a grout hole at the top of the lining was left open so that the imprisoned air could be expelled. The space was not being filled with liquid at high pressure; it was being filled with cement grout. Before it had been filled up to the top he was sure that the cement grout at the bottom had already more than half set. When the grout appeared at the top of the lining it was known that the space had been filled and the grout plug could be put in, but the annular space had only been filled at atmospheric pressure, so no compression stress had been put on the steel lining.

He would like to know if the new design mentioned on p. 450 was behaving even better than the Don-Seg segments were credited with having behaved.

Mr H. D. Manning said that the Paper was very interesting and valuable. The questions he wished to ask were not concerned so much with the experimental work as with the method of tunnelling. He would be glad if the Authors could elucidate their reference to the rate of construction. They had not made any direct reference to that in the Paper, but they did say that the length of about 2,000 ft had been completed in about 4 months. He had calculated that if work had been going on all the time, that would be at the rate of about 1 mile a year, which was about twice the rate normally expected by more orthodox methods of tunnelling. He believed that the actual rate achieved had been considerably greater than that, and he would be interested in any figures that the Authors could give.

Reference was made in the Paper to variations of line and level. The importance of variations obviously depended on the use to which the tunnel was to be put. He happened to be particularly interested in the possible use of tunnels as trunk sewers at very flat gradients, and level was a matter of the greatest importance. He would be glad if the Authors could give any figures for the permissible tolerances to which they considered that that form of tunnel lining could be laid both for line and level. He was particularly interested in the probable level variations, because a sewer invert had to be correct to within, say, ¼ in., and any likely variation in the segments had to be allowed for by enlarging the diameter of the segment ring to allow errors to be accommodated in the collar joint between the segments and the brick (or other) lining.

It was suggested in the Paper that if a tunnel were deep enough it could travel in a straight line between two points. He was afraid that the lawyers would not always allow that! The sort of tunnels to which he was accustomed very often had to wriggle about to follow street lines, and he would be glad to hear from the Authors what sort of radii were permissible and how such radii were formed—presumably by packings—and whether there were any particular difficulties associated with the work of negotiating bends.

His third question related to infiltration. Mr Segrave had made a reassuring reference to that matter, but Mr Manning was not sure that it satisfied him entirely, especially where the tunnel was in the upper strata of London Clay, where there were wide variations in the quality of the clay itself, or where seams of sandy strata were found. He would like to know whether any figures could be given for the actual infiltration. Had it been measured at all?

His fourth question related to the experiments, which he believed had covered a period of 1 year. He was not quite clear whether the experiments were still going on, and he would welcome the Author's views on whether they considered that the period which the experiments had so far covered was sufficient to be certain that a condition of stability had been established in the clay. In other words in a clay of the character described, the growth of pressure would be very slow. He was a little puzzled by the reference on p. 426 to the shear strength of the clay. No doubt he was simply exposing his ignorance of soil mechanics, but he had always thought of clay as a plastic material. From the note in the middle of p. 426 he had got the impression that the clay as cut would stand up indefinitely without any segments at all. That tied up with his question about the rate of development of pressure; he thought that it might well take many years to develop.

Reference was made on pp. 401–4 and in Table 2 to experimental linings. Apparently several experimental linings had been put in, but there was no information on what had happened to them, nor was it stated whether any information had been obtained from them.

Having asked those questions, he now wished to make a few comments on the Paper. The curve of costs given on p. 453 was very interesting, and it might be of interest to refer to some work done within the past year or so on more orthodox two-ring brick, three-ring brick, and concrete flange segments with brick lining in the London Clay.

For a 6-ft-finished-internal-diameter sewer the cost per yd excluding the lining ring of brickwork worked out at £62 per yd for brick construction and £75 per yd for flanged concrete segments. Those figures should be compared with £45 per yd shown in the curve for Don-Seg construction.

In conclusion, he felt it should be realized that that particular method of construction was suited only to good clay. London Clay varied very widely throughout the London area. He imagined that in driving a reasonable length of heading by that method, one must be sure (*a*) of good clay, and (*b*) of uniform clay, otherwise one would be up against difficulties of the size of the shell in relation to the size of the segments. Those difficulties were largely met by the rather cunning device of the alternative method of construction, the short key-segment. One must, of course, also have reasonably long runs of tunnel in order to cover fairly high mould costs and shield costs compared with more orthodox brick ring construction.

Mr R. A. H. Burton referred to p. 408, where the Authors said that consolidation-swelling tests had been made on several samples of the London Clay through which the tunnel had been driven and that the critical swelling pressure exerted had been found to be about 5·2 tons/sq. ft, which corresponded closely to the natural overburden pressure under certain conditions of pore-water pressure. Might it therefore be assumed that (*a*) the swelling pressure determined indicated that the clay was capable of exerting a radial pressure on the tunnel of the same order; and (*b*) that the tunnel was above the local ground-water table, since otherwise the intergranular pressure would have been reduced by the buoyancy produced by the hydrostatic head of water ?

Had the Authors any additional figures of the swelling pressures exerted by different types of clay in similar circumstances and, in particular, the clays of South-East England.

On the Southern Region of British Railways experimental data had been obtained concerning the swelling pressures exerted by two different clays, namely, a clay of the Fairlight series, a rather silty clay and very dense, near Sussex, and the other a Wealden clay from farther North. He would give a few details of those types of clay in order to identify them more closely. The Fairlight clay came from Ore, near Hastings, and had a liquid limit of 36–40, a plastic limit of 21–26, a plasticity index of about 15, a moisture content very much lower than the London Clay, ranging from about 5 to 10% in the in-situ conditions. The specific gravity of the Ore clay was 2·67 and of the Weald clay 2·69 ; the density of the Ore clay was 148 and that of the Weald clay 146 lb/cu. ft. The swelling pressure for the

Ore clay was 1·5 ton/sq. ft and for the Weald clay 3·5 tons/sq. ft. In the case of the Ore clay, which had come from a tunnel, the natural overburden of the tunnel had been about 150 ft but, since the sample had come from in front of the footings of the side walls it could be said that the immediate local overburden pressure had been only that resulting from about 3 ft of material. In the case of the Weald clay the sample had been newly excavated from about 22 ft below ground level.

It might be of interest to describe briefly the reason why the Fairlight clay had been sampled. Shortly after certain repair work had been carried out in Bo-Peep Tunnel—described in a Paper to the Institution by Campion[5]—it had been found that there were very slight inward wall movements occurring in the neighbouring Ore Tunnel about 1½ mile or rather more towards Rye and Ashford. Those movements had been of the order of about ¼ in. per year. There had been concern lest they might presage a further possibly catastrophic movement in the tunnel, and there had been anxiety to find out why they happened. The natural clay through which the tunnel was constructed was a very hard silty clay, and it had been found impossible—as the Authors had described—to sample it by ordinary means. The movement had taken place in an almost completely dry portion of the tunnel. There had been a little collection of water at the base of the lining round the footings, and the tunnel had no invert; that had contributed towards the inward movement.

It had also been found that the clay underneath the footings had softened to a strength of about ¾ ton/sq. ft. From that it had been deduced that a horizontal pressure on the walls of about 1·5 ton/sq. ft would be sufficient to cause a very slow inward movement. Further samples of the clay had therefore been obtained from near the footings—but it had not been possible to obtain those from behind the tunnel walls—and in as dense a condition as possible, during the course of excavating to place stretchers to give support to the footings. The strongest samples tested triaxially had been found to have a shear strength of about 3½ tons/sq. ft; somewhat harder samples had been subjected to consolidation-swelling tests similar to those described in the Paper, giving a swelling pressure of 1·5 ton/sq. ft. It had, in fact, been found (and it seemed to be significant) that that swelling pressure corresponded quite closely with the calculated pressure required to cause inward movement of the walls. Mr Burton could not feel sure that the true swelling pressure of the clay had been measured, and it seemed that, having regard to the fact that some of the figures given in the Paper, for London Clay, were several times as high, possibly the figures he had quoted for the Fairlight clay had been affected by the previous absorption of water in the neighbourhood of the tunnel footings.

With regard to the Weald clay sample, they had been a little puzzled by the fact that it was apparently capable of exerting a swelling pressure greater than the overburden to which it was subjected in situ. The overburden from 22 ft of overlying ground appeared to be about 1½ ton/sq. ft and the swelling pressure had been 3·5 tons/sq. ft.

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[5] F. E. Campion, "Part Reconstruction of Bo-Beep Tunnel at St Leonards-on-Sea." J. Instn Civ. Engrs, vol. 36, p. 52 (1950–51).

Dr Charles Jaeger said that it was not necessary to point out the importance of the comprehensive Paper and to say how much it was appreciated. He had himself dealt with similar problems in a recent Paper.[6]

The detailed study of such a Paper required far more time than was available, but a few general conclusions might be drawn from it.

First, a job of the type described was possible only in a rock which was naturally prestressed, the prestressing being of the type described by the theory of Heim.

Secondly, the success of the job clearly proved that some of the commonly accepted rules for the minimum thickness of the overburden could, in some cases, be neglected quite safely.

Dr Jaeger himself had reached similar conclusions for some hydro-power tunnels. Provided that the geological and topographical conditions permitted a clear analysis of the problem, the commonly accepted rules concerning the minimum overburden were too pessimistic.

It might be interesting to compare the Authors' Paper with a recent publication by Dr Frohnholzer on the Rosshaupten tunnel.[7] That concrete-lined tunnel was reinforced for part of its length with pressure grout according to the method of Dr Keiser. The reinforced section had been thoroughly checked before, during, and after grouting. The apparatus used for recording strains and stresses had been made by the German firm of Maihak. About 27,000 readings had been made and analysed.

Those two detailed Papers on rock prestressing had recently been made available to civil engineers. Many more were required to help to solve the difficult problem of rock behaviour in pressure tunnels. In 1950 Dr Jaeger had submitted to the World Power Conference that a committee should be formed for the study of underground power stations. The principal task of such a committee would be to foster the development of "Rock Mechanics" similar to "Soil Mechanics," the tremendous developments in which civil engineers had been watching for about 35 years. The considerable development of underground hydro-power stations was a justification for that proposal.

That idea had also been tentatively submitted to engineers in Britain and abroad, and the conclusion that something should be done on those lines had generally been accepted.

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[6] Charles Jaeger, "Present Trends in the Design of Pressure Tunnels and Shafts for Underground Hydro-electric Power Stations." Proc. Instn Civ. Engrs, Pt. I, vol. 4, p. 116 (Mar. 1955).

[7] Dr J. Frohnholzer, "*Messungen am Hauptstollen des Lechopeidurs.*" Rosshaupten, Munchen (Mar. 1953).

He felt that practical steps should be taken to form a committee on rock mechanics and underground power stations.

Mr V. H. Collingridge, said that in the Conclusions to the Paper there was a statement that after about a year the vertical diameter of the tunnel had decreased by 0·06 in. and the horizontal diameter had increased by 0·02 in. Earlier Mr Tattersall had given as one of the purposes of using that type of lining the fact that it was prestressed in the nature of the method of its building. He rather felt that conclusion (5) went to show that that wish had not been completely realized.

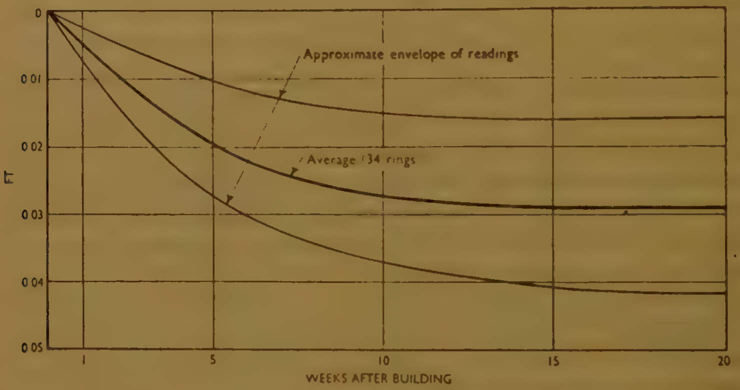

Fig. 34.—Measured downward displacement of crown
(25′-0″-int.-dia. cast-iron tunnels in London Clay with 80 ft overburden)

When a tunnel was excavated by conventional methods, it took some time for the load to come on the skin of the tunnel, and Fig. 34 illustrated that. It showed figures in regard to a very much larger tunnel, but those were the actual measurements taken over a long period. The figures showed that it took about 3 months before the lining settled down. He thought that in that case it had taken about a year for the Don-Seg lining to settle down to shape, and he felt that that went to show it was not fully prestressed. That might be due to the fact that variations in the clay itself, whether sandy or silty, resulted in minute gaps at various places between the back of the lining and the clay itself, which got filled up in the course of time by the deformation of the lining.

That raised another point. Would it not be a very good insurance policy when using that type of lining to have a hole cored in each segment and give it a very small injection of cement grout under high pressure so as to make sure that any voids which existed were at once filled ? The cost would be very slight, and it would ensure that a complete job had been made of the lining.

⁂ **Mr E. W. Cuthbert** said that the engineers of the Metropolitan Water Board deserved great credit for appreciating the possibilities of the plain Don-Seg tunnel lining as a water conduit, particularly since that lining had been chosen originally as a cheap form of primary lining to be used in conjunction with a steel secondary lining. The proposal to omit the steel lining from the Thames–Lee main had been made before the experimental work at Ashford Common was far advanced. After long discussions it had been decided to omit the steel, except where the main passed through access shafts and crossed the line of some deep tunnels of other authorities.

In Mr Cuthbert's opinion three main requirements had to be met if risk of excessive softening of the clay accompanied by collapse of the lining was to be avoided. Those were; first, that the segments should be accurately made and erected; secondly, that no appreciable cavities should be left behind the lining and; thirdly, that the circumferential joints should be filled. Careful supervision was therefore essential.

The Authors' views on errors of line and level had not Mr Cuthbert's unqualified agreement for the following reasons.

Some relaxation of accuracy of line might be permitted but, since most tunnels had to be driven between fixed points, any large accumulated error in line would ultimately have to be corrected by the use of packings in the circumferential joints of a large number of rings of the lining. Furthermore, if the line of the tunnel as constructed deviated appreciably from the designed and approved line which might well appear on plans submitted to local authorities, owners of properties, and others, the professional reputation of the constructor would suffer.

The Thames–Lee main would be constructed on gradients as flat as 1 in 1,000 for considerable lengths. With the best possible care it would be difficult to avoid local reversals of gradient, and any relaxation would almost certainly lead to large pools being left in the main after dewatering.

For the foregoing reasons Mr Cuthbert thought that only errors of line could be allowed to exceed the 1 in. usually specified for tube-railway work, but that they should not be allowed to accumulate in the way the Authors suggested.

It appeared from the diagram of the radial pressure gauge given in Fig. 3 that when the tunnel and the space around the gauge were filled with water the gauge would register the difference between the radial earth pressure on the back of the loading plate, and the internal water pressure acting on part of its area. The radial earth pressure on the back of the plate would be expected to remain almost unaltered until the initial hoop compression was relieved, after which it would increase linearly with the internal water pressure until failure took place. It was suggested that the ordinates in Figs 28, 29, 30, and 31 described as "radial earth pressure"

⁂ This contribution was submitted in writing upon the closure of the oral discussion.—SEC.

should be re-titled " difference between radial earth pressure and internal water pressure."

The Authors rightly stressed the importance of getting the correct relation between the diameters of the shield and the lining. The best width for the closing segment was probably determined most conveniently by trial in the tunnel, after which the manufacture of the closing segment slightly shorter than the other segments should give sufficient adjustment to meet local variations in the ground.

The Don-Seg and wedge-block linings had been made with the tapered longitudinal joint faces other than plane. In the Don-Seg lining the joints had been made radial to the axis of the tunnel throughout their length, but in the wedge-block lining the edges of the closing block only were radial to an axis offset downwards from the axis of the tunnel. That feature was presumably intended to ensure that adjacent segments did not move radially relative to each other during the closing operation. Surface contact between all adjacent longitudinal joint faces could occur only when the lining had been expanded to exactly its correct diameter. In Mr Cuthbert's opinion the segments might well be equally satisfactory if the longitudinal joint faces were made plane and he suggested that tests should be made to check that. The substitution of plane surfaces for curved surfaces would lead to savings in mould charges and surface contact would then be obtainable at all stages of the closing operation.

The Authors, in reply to Mr Bulman, said that a more thorough study of the Paper would have clarified many of the points which he had raised. He had suggested that there was something wrong with the statement on p. 404 that the pressure would require a depth of 59 ft. That figure was quite correct. The head could rise to about 100 O.D., and the ground level was approximately 41 O.D., which meant that the water level could rise to 59 ft above ground level. If the specific gravity of the clay was 2 then it was necessary to put the tunnel as far below ground as the water pressure could rise above ground and the figure was 59 ft.

With regard to the hoop tests, it had been asked why the friction did not work both ways. The answer was that it did. A jack load of 99 tons had been put into ring 981 at an angular distance of 144° from the hoop gauges, and that had given a hoop-gauge reading of only 35 tons. Upon removing the jack load, the thrust in the ring at the jacking point had fallen to zero, whilst the thrust at the gauges had fallen only to 27 tons. In that condition the ring was held in place entirely by frictional and adhesive forces.

Mr Bulman had then asked about friction on the adjacent rings. That also had been investigated but it had been found that applying a circumferential thrust to the outer rings had no significant effect on the behaviour of the centre ring. The Authors had thought for some time that there might be considerable friction between rings because the whole tunnel was continuously prestressed longitudinally, but of course the load was removed from every ring and apparently it did not build up.

There were certainly ways in which the present "Don-Seg" design might be improved. The closing pressure was very erratic and sometimes very high. That gave rise to high local pressures and considerable spalling took place. Also it was often difficult to close the rings completely and that created gaps at the circle joints. It was wrong to think the ring was tightened at five points, for the ring was quite loose when the first projecting segments were driven home. The hoop thrust did not build up until the last segment was nearly home, so that the hoop thrust was still effectively applied at only one point.

The improved design referred to in the Paper appeared to have met those points. A length of tunnel using the same design of lining was now being driven and it had been possible, for the first time, to close the rings to a specified ram thrust. That was essential if prestressing was to be relied upon. The longitudinal joints fitted much better, the rings could be easily closed, and consequently the circle joints were without gaps. All those factors contributed to a more uniform stress distribution and there was no spalling of the concrete. There was also no tendency for the key to rise into the clay.

Since each ring contained only one wedge, the backs of the other segments could be lubricated to an extent which, if applied to the wedge, might cause it to be forced out when the ram was withdrawn. The soft-soap lubricant was applied by brush to the surface of the clay and there was no doubt that it gave a more uniform hoop thrust. No one would willingly put double the load in a segment if he could avoid it. The main concern, from the prestressing point of view, was the minimum load, and every effort should be made to avoid the necessity for putting in a maximum load double that minimum load to cater for the friction.

The Authors had not stated that the shield had deformed ½ in. They had in fact never measured the shield in the tunnel. When the shield was taken out of the tunnel it was circular and it was not known whether it had been deformed in the tunnel or not. It was known that the vertical diameter of the tunnel was about ½ in. greater than the horizontal diameter and the Authors were of the opinion that that was probably caused by the greater compressibility of the clay vertically across the bedding planes, rather than by deformation of the shield.

Mr Seagrave had also commented on the diameter measurements. The Authors agreed that the changes in diameter were very small when compared with the constructional tolerances on the shield and the lining. They were, however, of extreme importance in deciding whether an inner steel lining would be necessary and in determining the behaviour of the inner cement rendering or gunite lining which might eventually be applied.

In answer to a point raised by Mr Manning, the experimental cement inner linings detailed in Table 2 were applied to investigate their behaviour under the working conditions of the tunnel, and they had all behaved perfectly.

The diameter observations quoted by Mr Collingridge in Fig. 34 should not be compared with the Ashford Common tunnel. Not only was the ratio of depth to diameter quite different, but in that case there were several tunnels side by side, and that must have had a pronounced effect on the stress distribution in the clay and tunnel lining.

Mr Manning had asked about the rate of tunnelling. The rate had been 10 ft 6 in. per shift. Working two shifts per day and five days per week gave 105 ft per week, or almost exactly one mile per year as Mr Manning had calculated. That had been accomplished very easily, once the men had become accustomed to the work, and the rate could probably have been increased if necessary.

The question of tolerance and accuracy obviously depended on the use to which the tunnel was to be put. In the case of a sewer it was obviously desirable that the gradients should be uniform. But in the case of a pressure tunnel those considerations did not apply except that, as a matter of convenience, it should be possible to drain the tunnel as easily as possible.

The accuracy of both line and level depended entirely on the accuracy with which the shield was driven, since the lining had to occupy the hole cut by the shield. If necessary a tolerance of ½ in. could probably be kept.

The radius of the curve in the Ashford Common tunnel was 500 ft and ordinary segments were used with packings. Probably 200 ft would be about the minimum radius that would be driven with ordinary segments and for smaller radii special segments would be necessary.

Mr Bulman seemed to doubt the reliability of some of the instruments that had been used. All the radial earth-pressure gauges had been unloaded, as explained, and the errors in the zero readings were given in the Paper; they were extremely small, and, so far as was known, they were probably the most stable remote-reading load-measuring instruments for which records had been published.

Mr Cuthbert had experienced some confusion concerning the various pressures acting on the tunnel when charged with water. The Authors agreed that at first sight the conditions seemed complicated and they had taken considerable care to select the best designations for the various pressures.

In considering the pressures acting when the tunnel was flooded, two alternatives were possible, either that the lining was pervious and the water pressure acted directly on the clay, or that the lining was watertight with the water pressure acting on the inside of the lining. In the relevant sections of the Paper it had been assumed that the lining was in fact pervious.

The design condition was that the hoop thrust in the lining should remain compressive under the action of the internal water pressure. Hoop thrust depended only on the net (or resultant) radial pressure acting on the lining.

For a pervious lining, the net pressure was the radial earth pressure on the lining, and for an impermeable lining the net pressure was the difference between the internal water pressure and the external radial earth pressure acting on the lining. Simple theory showed that for any given internal water pressure, the net pressure on the lining was the same for either alternative.

The design of the radial earth pressure gauge (Fig. 3) was such that the space round the load plate became flooded and the gauge directly measured net radial pressure on the lining, or, in the terminology used in the Paper, the radial earth pressure on a pervious lining.

The actual factor of safety against the lining becoming unloaded was difficult to estimate. Equation 3 of the simplified theory given on pp. 446 and 447 showed that the critical water pressure which would cause the lining to become unloaded was about 1·4 times the average radial earth pressure on the lining with the tunnel empty. Taking an average value of 3 tons/sq. in. the critical head would be about 150 ft above axis. However, during test 2 the tunnel had been subjected to 160 ft head without any signs of the lining having become unloaded, and the change of slope of the curves in Figs 30 and 31 suggested that the simplified theory did not hold at large heads. Extrapolating those curves would indicate a critical head of as much as 220 ft and further investigations of that point would be carried out when the conduits supplying the tunnel were complete.

The Authors agreed that the general results of the experiments on the London Clay tunnel were in agreement with current ideas on the behaviour of high pressure tunnels in rock discussed by Dr Jaeger in his recent Paper.[8]

Mr Manning had asked a number of interesting questions. The long-term effect had not been mentioned in the Paper for the very good reason that it was a factual Paper, and the long-term effects were a matter for speculation. The Authors' general view was that whatever the long-term effects proved to be, they would not entail any risks. There was at the moment a load on the lining which was less than existed on any of the London tunnels on which measurements had been made. The Building Research Station had recently been making a preliminary study of a group of observations on a tunnel in London which was 30 years old and the stress in the lining was equivalent to about 0·8 of the overburden pressure. If 0·8 of the overburden pressure developed at Ashford Common, the margin of safety would be even greater than it was today. It was expected that the pressures on the lining would increase, but there had been little significant sign of it so far. Observations were still being made and it was hoped to continue them as long as possible.

So far as they knew, the clay at Ashford Common was stronger than any other London Clay which had as yet been tested. The face of the tunnel, if it were unsupported, would probably not collapse for a number of years.

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[8] See reference 6, p. 455.

Mr Manning had also asked about water pressure in the London Clay. At the moment a general statement was not possible, for the water-pressure distributions seemed to vary from one site to another. It was well known that in some parts of the London area the water had been drained from the underlying chalk. Such changes and even changes in water level that had occurred thousands of years ago would still be influencing the pore pressure in the London Clay, especially at Ashford Common where the clay was more than 400 ft thick. The present indications from various tunnel excavations were that in some parts of the clay under London there was a water pressure and in other parts there was not.

There had been an attempt to measure the local pore-water pressure a few feet from the tunnel at Ashford Common just after construction, but it was quite difficult to measure pore pressures in such a strong impervious material, and the Authors had not been convinced by the observations. A pressure had been measured at one point and a suction at an adjacent point. Probably the technique had been in error.

In the absence of adequate information on the water-pressure conditions deep in the clay beneath London, the long-term changes around the tunnel could be estimated only on the basis of a number of assumptions. One simple assumption was that the initial water-pressure distribution at Ashford Common was hydrostatic relative to the water level in the superficial gravel. Then the head of water in the clay at the level of the tunnel before excavation would be about 86 ft. In the course of digging, the water pressure at the tunnel wall was reduced to atmospheric pressure and hence water would slowly drain into the tunnel, which in fact, it did. The infiltration over the entire length of the tunnel was too small to measure by normal methods. Some idea of the quantity might be gained from the description on p. 416.

So long as the tunnel remained empty, the reduction in water pressure would cause the surrounding clay to consolidate and strengthen, but that process might take hundreds of years to reach equilibrium. A short time after construction the tunnel had been filled with water and the tunnel would operate under a water pressure which was greater than the initial assumed water pressure. Hence the clay would commence to swell and soften around the tunnel. That process would have two desirable effects, the softening would tend to eliminate the present irregular stresses in the lining and the swelling would lead to an increase in the average thrust in the lining, probably up to a limit equal to the full overburden pressure minus the water pressure in the tunnel. The total amount of swelling would be quite small and would take a long time to complete.

Mr Burton had raised some points concerning the consolidation–swelling tests which had been made on the London Clay. It so happened that the average swelling pressure of 5·2 tons/sq. ft was about equal to the present total overburden pressure. There were considerable technical difficulties in measuring those swelling pressures, but the experimental result was presumably a guide to the present effective pressure in the ground.

It should not be assumed that such a thick stratum as the London Clay was in a state of equilibrium. The present effective pressure at the level of the tunnel was probably related to changes in overburden and water-table which had occurred thousands of years ago.

The ultimate pressure developed on the tunnel lining depended not only on the swelling pressure of the clay at the time of construction, but also on the volume and stress changes subsequently occurring throughout the stratum. The tunnel had not been empty long enough for those effects to reach equilibrium.

The closing date for Correspondence on the foregoing Paper has now passed without the receipt of any communication.—SEC.

JOINT MEETING WITH THE INSTITUTION OF ELECTRICAL ENGINEERS

6 January, 1955

David Mowat Watson, B.Sc., President I.C.E., in the Chair

The following Paper was presented for discussion in the Lecture Theatre of the Institution of Electrical Engineers and, on the motion of the Chairman, the thanks of the two Institutions were accorded to the Authors.

Paper No. 6042

PROBLEMS OF HYDRO-ELECTRIC DESIGN IN MIXED THERMAL–HYDRO-ELECTRIC SYSTEMS

by

***T. G. N. Haldane, M.A., M.I.C.E., Past-President, I.C.E., and**
P. L. Blackstone, T.D., M.A., M.I.E.E.

SYNOPSIS

The Paper deals with some of the problems which arise in the planning and development of a system comprising both thermal and hydro-electric plant. In general, when thermal plant predominates it is considered desirable to develop hydro-electric power with the greatest installed capacity the system can absorb in conjunction with the available amount of energy. Practical limitations to low-load-factor operation are discussed.

The effects of the introduction of low-load-factor hydro-electric plant on system operation and on the overall fuel consumption of the thermal plants are examined. New circumstances which cannot readily be forecast may later make desirable a change in the design load factor of some of the hydro-electric plants, and the Paper stresses the advantage of having, where possible, flexibility in the initial design to enable such changes to be made. Where circumstances permit, consideration of the use of pumps, and especially reversible pump-turbines, is recommended as a means of augmenting storage capacity and either increasing the firm annual load factor or permitting increased installed capacity.

The basic economics of the pumped-storage scheme are explained and an example illustrating the flexibility which is possible in planning such an installation is given in the Appendix.

The Paper discusses the effect on hydro-electric development of the possible ultimate replacement of coal and oil by nuclear fuel in thermal plants, and it is concluded that hydro-electric plant is likely to continue to be advantageous for peak-load operation and that there is no justification for any postponement of long-term investment in hydro-electric works on account of the advent of nuclear power.

The effect of rising prices is also discussed, and a general survey is given of present practice in the design of hydro-electric machines. Transmission practice and problems are briefly reviewed as an integral part of hydro-electric design.

In conclusion it is pointed out that differences of opinion between thermal and

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* The Authors are with Messrs Merz and McLellan.

hydro-electric engineers may arise because of inadequate mutual understanding of the inherent characteristics of the two types of plant and of the problems of combined operation. The design of hydro-electric projects in a mixed system must be based, not only on the hydrographical and civil engineering data, but on a comprehensive study of all the many factors involved.

(1) INTRODUCTION

DURING the last quarter of a century the development of hydro-electric resources has increased approximately fourfold, the total world installed capacity of hydro-electric plant being now about 90 million kW. This rapid development is in part due to technological advances, particularly in high-voltage transmission, which have made it possible to harness on an increasing scale hitherto unused resources, some of which are very distant from the load supplied. In part, also, the rapid development is due to rising costs of fuel and shortages of indigenous fuel in certain countries. Although a threatened shortage of indigenous fuel may greatly affect the national power policy, the cost of thermal generation is the main criterion in deciding whether any hydro-electric project is, or is not, worth constructing.

Thermal and hydro-electric plant are to a considerable extent complementary, and advantage is usually obtained from a mixed system. This mixture of plant has created certain interesting design problems which are examined later. Table 1 shows the proportions of thermal and hydro-electric plant in various countries of the world, and it will be noted that this proportion varies over a wide range.

TABLE 1.—APPROXIMATE PROPORTIONS OF THERMAL AND HYDRO-ELECTRIC PLANT IN VARIOUS COUNTRIES *

	Thermal %	Hydro-electric %
England and Wales	99·7	0·3
Australia	88·1	11·9
Germany (Western)	81·1	18·9
United States	77·8	22·2
Scotland	69·4	30·6
France	55·5	44·5
Finland	39·8	60·2
Japan	37·4	62·6
Sweden	22·7	77·3
Italy	18·3	81·7
New Zealand	11·5	88·5
Canada	9·6	90·4
Switzerland	6·6	93·4
Norway	4·2	95·8

* Source: *United Nations Bulletin of Statistics*, November 1953.

In examinations of the combined use of thermal and hydro-electric plant in large integrated systems there are two concepts which are commonly used. The first is the annual load/duration curve with the various types of plant fitted into the area embraced by the curve; an idealized diagram of this type is shown in Fig. 1. Although such diagrams can be of help, they have considerable limitations and unless used with caution may lead to erroneous conclusions. Before decisions about a hydro-electric project are reached it will be necessary to make more detailed studies on a daily or short-period basis, and a method of doing this is described in Section 2.2.

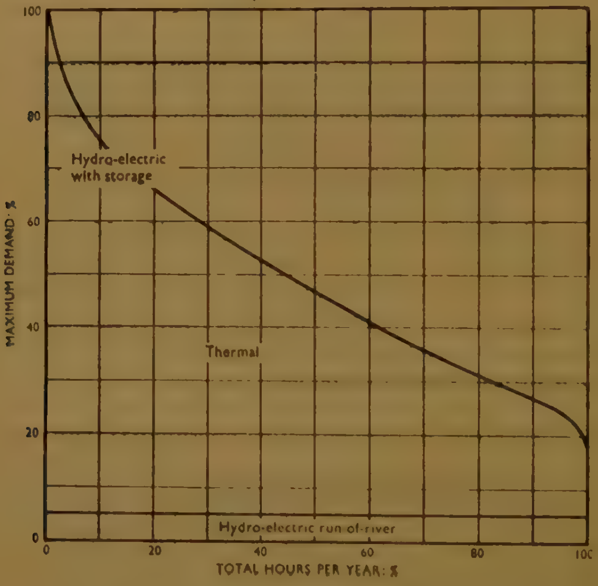

FIG. 1.—IDEALIZED LOAD/DURATION CURVE

The second concept is that of the annual load-factor of hydro-electric stations, although this is somewhat ambiguous and should also be used with caution. The firm annual load-factor of a hydro-electric project can be defined as the ratio of the energy output which can be relied on in any year (including very dry years) to that which would be generated if the installed plant operated at full load throughout the year. The average annual load-factor of a hydro-electric project is the similar ratio for the average output over a long period of years, and therefore includes a certain proportion of non-firm energy, i.e., that which is not available in the dry years. In any given hydro-electric project the available energy is more or less fixed by the run-off, the principal variable being the installed capacity of plant. Hence changes in the average annual load-factor of a particular project usually mean changes in the installed capacity.

An approach to the study of the optimum installed capacity of a hydro-electric project calls for many data additional to the hydraulic features of the particular scheme. First, it is necessary to consider and bring into account the transmission system. Secondly, it is necessary to study the system demand and load curves, both at the present time and as estimated for the future. Thirdly, it is necessary to consider what other hydro-electric projects are likely to be developed in the future and how they can best be fitted in with the project under immediate consideration. Such studies require to be combined with corresponding studies of the complementary thermal plant.

A full investigation therefore presents a complex problem, and in practice it may be feasible to achieve only approximations to optimum design. It remains, however, highly advantageous to obtain the most accurate approximations possible with the available data.

(2) DEVELOPMENT OF A COMBINED HYDRO-ELECTRIC AND THERMAL SYSTEM

(2.1) *The problem of installed capacity*

The capital cost of hydro-electric power can be considered as made up of two parts: expenditure upon the main storage works and associated aqueducts, and expenditure on tunnels, pipes, buildings, plant and transmission. The storage works are not directly dependent upon the installed capacity of the plant and may in the simple analysis be regarded as independent of the maximum rate of flow through the machines. The tunnels and pipes are designed to pass the full-load flow required by the machines, and their cost is thus directly connected with the selected plant capacity. The capital or annual costs of a particular installation therefore consist of a fixed amount representing storage and an amount which varies with the plant capacity.

The cost of storage usually forms a large proportion of the total cost of a hydro-electric installation, and the capital cost per kilowatt decreases as the amount of installed plant is increased. It will be found that for most projects the incremental cost of increasing the plant capacity, together with the provision of larger conduits and transmission, is less than the capital cost of increasing the installed capacity of a thermal station.

In countries where hydro-electric power resources predominate and where fuel is costly, it is probable that the comparison of generation costs will favour a hydro-electric project over a wide range of installed capacity. Decisions regarding the design of the hydro-electric stations will nevertheless involve a detailed study of the estimated cost of production and transmission of power for different installed capacities at the various possible sites. Such a study is, of course, subject to the overriding consideration that the total firm power and energy from the hydro-electric plant and any thermal plant can be satisfactorily combined so as to meet practical operating requirements.

If the seasonal variations in run-off do not conform with those of the power demand, as for example in a country where the water is derived largely from melting snow in spring, or where a heavy demand for irrigation pumping arises in dry weather, or where long-period droughts can occur, it may be that the storage required to produce a reasonable proportion of firm energy is either impracticable or excessively costly. Additional thermal plant may then have to be added to cover the deficiencies of the hydro-electric stations, and this will be operated (probably on a base-load regime) in times of water shortage. Some thermal plant is nearly always essential for full exploitation of hydro-electric resources and has been found necessary even in countries particularly well endowed with water power.

A problem of special interest from the planning point of view arises where a large proportion of the plant is necessarily thermal. With continued growth of load all countries will increasingly tend towards this condition, and further consideration will therefore be confined mainly to such circumstances, although the principles involved apply also to other conditions.

In a predominantly thermal system it will usually be preferable, for the reasons already given, to develop such water power as can economically be justified in comparison with thermal power with the greatest installed capacity, i.e., the lowest firm load-factor which the system can accept in conjunction with the more or less fixed amount of energy. Furthermore an increase of plant capacity permits the spilling of water at times of high run-off to be reduced, resulting in better utilization of the available water and economy in the use of fuel by the thermal plant.

(2.2) *Practical limitations to low-load-factor operation*

In a purely thermal system the machines will, so far as possible, be brought into operation sequentially in ascending order of their fuel cost per kilowatt-hour sent out, which generally results in the oldest and least efficient machines running on peak load only. In terms of the load/duration curve this means that in an expanding system new high-efficiency plants will be brought in at the bottom, making all older plant take progressively higher positions in the curve, until eventually each becomes peak-load plant. So long as the increase of efficiency of thermal plant continues, this process results in a maximum reduction in the average fuel consumption for the system.

It is not possible, however, to achieve the theoretical optimum use of plant in practice ; factors such as reduced availability of base-load plant, lack of flexibility of peak-load plant, transmission limitations and operating difficulties (such as a tendency to run up the peak-load plant sooner than

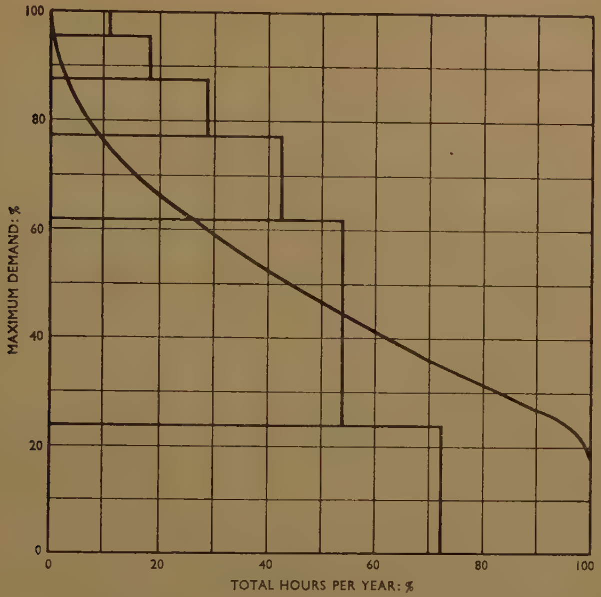

FIG. 2.—TYPICAL ANNUAL LOAD/DURATION CURVE SHOWING ENERGY GENERATED BY DIFFERENT BLOCKS OF PLANT

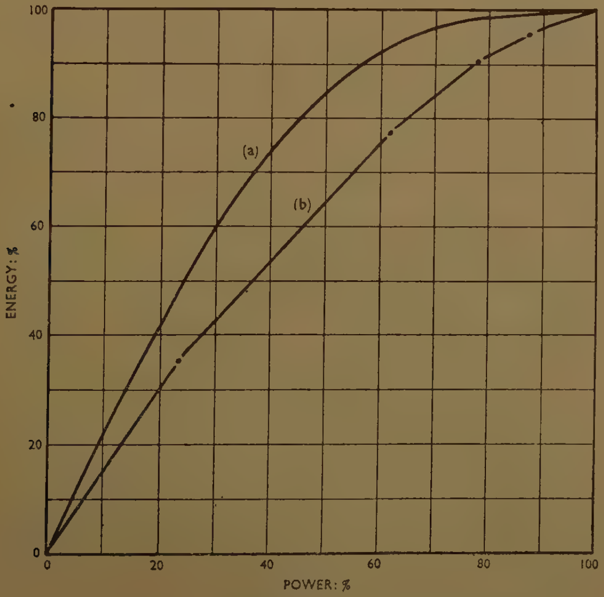

FIG. 3.—INTEGRATED DURATION CURVES

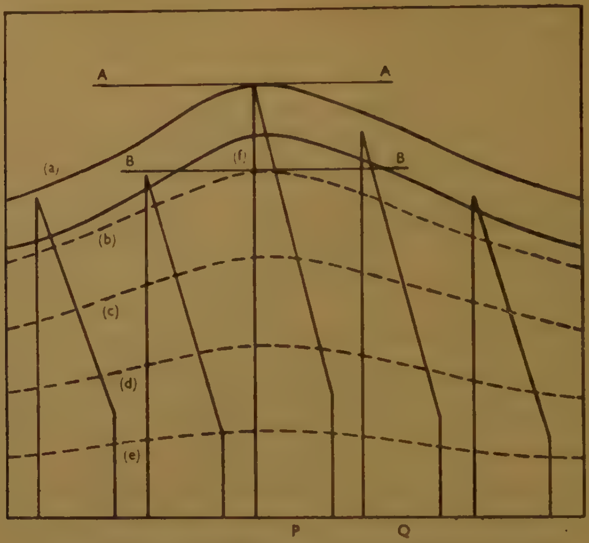

FIG. 4.—BASIS OF ADJUSTED LOAD/DURATION CURVE

(a) Maximum demand (m.d.).
(b) 80% m.d.
(c) 60% m.d.
(d) 40% m.d.
(e) 20% m.d.
(f) Minimum demand.

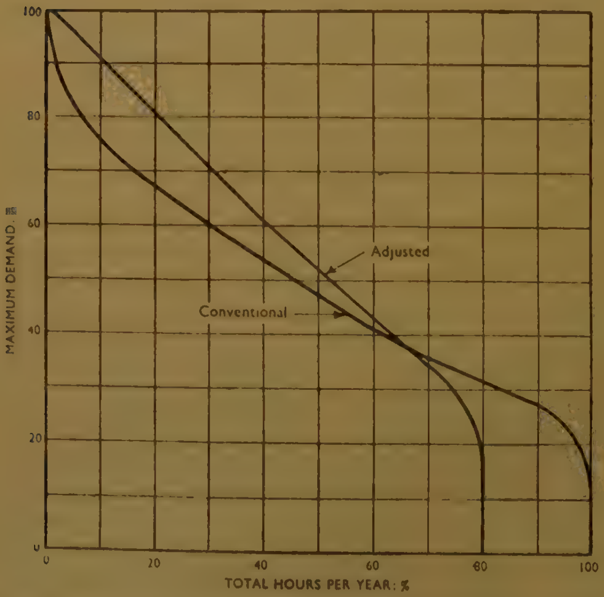

FIG. 5.—CONVENTIONAL AND ADJUSTED LOAD/DURATION CURVES

is necessary) result in the less-efficient stations generating more than their theoretically desirable quota and the more-efficient stations generating less. Fig. 2 shows a typical annual load/duration curve, with superimposed on it the blocks of energy generated by different plants grouped in order of fuel costs per kilowatt-hour sent out. The problem can also be illustrated by integrated duration curves (Fig. 3), the slope of which is proportional to the load factor of the various blocks of power. Curve (a) corresponds to the load/duration curve with the ideal operating condition of the base-load machines generating their full desired quota of kilowatt-hours. Curve (b) represents what is actually achieved in practice, as indicated by the rectangles in Fig. 2.

For the purpose of settling the maximum practical installed capacity of a hydro-electric project, the conventional annual—or even monthly—duration curve is inadequate, as it gives no indication of day-to-day load conditions which the plant must be designed to meet; it is necessary to consider daily duration curves. In Fig. 4, P is the day of annual maximum demand and the remaining curves represent other typical days of the year. Some of such typical curves would refer to extremes of peak load and others to more favourable conditions obtained in better weather, the probable range of variation being determinable statistically. The normal type of annual duration curve is produced by integrating the areas of the individual daily curves enclosed by a series of horizontal lines such as AA and BB. Only on days P and Q does any load fall within these limits, and if a duration curve produced in this way were used as a guide to system operation the conclusion would be that some plant ought to run on those days only and be shut down on all other days of the year. This is not possible in practice and hence is misleading as a basis for judging results or forecasting the future.

There is no definite criterion for deciding how much energy the plant which generates a particular element of load on the day of annual maximum demand should generate on other days, but integration between curves such as (a) and (b) is a reasonable assumption. This implies that peak-load plant which carries the top $x\%$ of the load on the day of annual maximum demand should be capable of carrying the top $x\%$ on any other weekday when weather or other conditions lead to an extreme peak for that time of the year. Fig. 5 shows a duration curve of the conventional type, and superimposed is the adjusted curve of the type obtained by integrating between the curved lines in the manner illustrated by Fig. 4.

The importance of the adjusted curve in Fig. 5 is that it provides a more realistic starting point in assessing the lowest acceptable firm annual load-factor (or the maximum acceptable installed capacity) for any given block of energy. It still remains necessary, however, to allow in each particular case for factors such as transmission limitations and lack of operating flexibility, which tend to increase the acceptable load-factor. The latter will be of less importance with hydro-electric peak plant than with thermal plant.

(2.3) *The introduction of low-load-factor hydro-electric plant*

(2.3.1) *System operation.*—Hydro-electric plant has a higher availability than thermal plant, even though this is reduced to some extent by the longer transmission which is usually involved. It is more flexible because it can be started, synchronized, and brought up to full load much more quickly. For example, the whole capacity, 102 MW, of the Galloway scheme in Scotland can be brought on in about 15 min, even if the call is made when the penstocks and turbine casings are empty and even though this capacity is contained in eleven machines. The time is less if the casings are already full. It is possible to change load rapidly, and in consequence the load variations can be followed more closely by the peak-load plant.

Hydro-electric plant, therefore, is especially suited to rapid load fluctuations, and its use in this way benefits the operation of the system as a whole. The thermal plant will be working on a more constant load without sudden fluctuations and this makes its operation easier. It must be remembered, however, that achievement of the design load-factor of hydro-electric plant is dependent on sufficient storage being provided to regulate annual flow ; i.e., the assumed hydro-electric output must be firm, otherwise it might be necessary for thermal plant to be brought in at certain times to cover a deficiency of the water power, and the latter could not then be regarded as fully replacing an equivalent thermal capacity. In wet weather the water power can be generated at a load factor higher than that designed, thus saving fuel. Consequently, in a combined system, temporary departures from design operation will be desirable and will result in overall economy, whereas any departures in the all-thermal case produce an overall loss.

(2.3.2) *Overall fuel consumption.*—The introduction of hydro-electric plant at the top of the duration curve has four important effects on the total fuel consumption on the system :—

(a) The fuel which would have been needed to generate the actual hydro-electric output is saved.

(b) The fuel which would have been needed, owing to banking or other standby losses, to keep the equivalent thermal plant available for peak-load operation is saved.

(c) The quality of operation in the peak region is improved, because the hydro-electric plant is capable of following more closely the moment-to-moment variation in demand. With thermal plant it is more difficult to restrict the inefficient plant to minimum generation, and there is a consequent tendency to generate too much from the least efficient, and too little from the more efficient, plant. It may, in fact, often be difficult to restrict a peak-load thermal station to an annual load-factor of less than about 15%.

(d) There is a temporary loss of the chance to improve the average thermal efficiency which results from the addition of new thermal plant, whose efficiency would be higher than the average of that existing.

Factors (a) and (b) can be fairly closely calculated and (c) can be estimated, but (d) is more complex. To assess the ultimate effect it is necessary to study two alternative systems, one all-thermal and the other with a proportion of water power taking its place at the top of the duration curve. The analysis has to be in considerable detail and taken over a period of 25 or more years. The accuracy with which this can be done depends on ability to forecast the rate of system load growth, the annual improvement in efficiency of thermal plant and the trend in the price of coal; it also depends on assumptions which have to be made as to the life of thermal plant before it is removed from service as obsolete. Although such an analysis may not be feasible except as a rough approximation, it should be possible to reach a reliable conclusion as to the existence of an operating economy or loss, if not its magnitude.

(2.4) *Freedom to modify design load-factor*

The annual load-factor of a thermal plant can be varied as desired within certain limits, but the design load-factor of a hydro-electric installation (which fixes not only amount of plant but also size of tunnels and pipelines) must be determined as a result of economic studies before the scheme is designed. New circumstances which cannot readily be forecast may make a change in design load-factor desirable, and any freedom to do so in the future may be of considerable value. It will not usually be possible to reduce the load factor, i.e., to increase the plant capacity, unless some special provision is made when the scheme is designed and constructed. If the resources of a country are such that thermal plant must predominate, a hydro-electric plant designed for a particular load-factor will gradually tend to take a lower position on the duration curve as the system develops. There may be justification later for its position being raised, or at least kept where it was initially, by an increase in the plant capacity.

In low-head installations where the power station is at the foot of the dam, there may be comparatively little difficulty in constructing the foundations, short penstock and intake, and if desired, an extension to the power-station building for further machines to be installed later. With higher-head plants, where a tunnel, surge shaft and one or more pipelines are required, there is not the same scope for later extension of capacity and the difficulty and cost of constructing a new tunnel and pipeline may make the provision of additional plant impossible. The more economic course would probably be to construct all the water conduits for a larger capacity than it was intended to install initially. The incremental cost of doing this could be expected to be small relative to the incremental cost of thermal plant, and under the circumstances here considered it would be wise to look well ahead at the planning stage and consider what, if any, provision should be made for increase of capacity as the system expands. The particular case of the Tennessee Valley Authority may be mentioned as illustrating such a method of development. In the first 10 years the Authority installed 1,000 MW of hydro-electric plant, but during the second 10 years they installed a further 1,500 MW of hydro-electric plant and 1,500 MW of thermal plant.[2] The Authority have almost reached the limit of their potential and economic water resources, and to meet heavy new demands arising from a large atomic-energy project and other industries they are now constructing base-load thermal plants and have installed extra machines in spare bays provided in many of the existing hydro-electric plants.

(2.5) *Increase of firm hydro-electric power*

The proportion of the total mean annual output of a single hydro-electric plant or group of plants which can be regarded as firm, i.e., guaranteed in the driest year, is dependent on the amount of storage provided. The balance of the mean annual output is secondary energy which has a smaller value. Fig. 6 shows, for a typical annual load curve, the percentage of the mean annual output which can be regarded as firm for a particular storage capacity ; the curve relates to typical conditions of run-off in Scotland and would not necessarily be applicable in other countries. If the whole of the annual output could be made firm the maximum benefit would be obtainable, but it is evident that above about 80% firm output the amount of storage required rises very rapidly, and an increase beyond this point is possible only where exceptionally favourable conditions exist. Thus for any particular reservoir site there will be some firm output which it is not economic to exceed by increasing the size of the dam.

Where storage is limited this may, in effect, mean that a certain acceptable load factor can be guaranteed throughout part of the year but not all of the time. Either the doubtful period must be eliminated by reduced installed capacity, or some means of firming must be provided. Two such means are possible, namely the installation of thermal plant especially for firming the water power and the use of pumps. Either method can be regarded as a means of augmenting the effective capacity of a reservoir, thus increasing the percentage of firm power, or as a means whereby the reservoir capacity can be reduced for a particular amount of firm power.

The use of thermal plant for firming is expensive, but may be justified ; the use of storage pumps is a relatively new development, but considerable attention is now being given to this possibility, particularly in America. Of course, essential requirements are a tailrace pond from which to pump and a source of off-peak energy for pumping. An energy loss of some 33% will be incurred, but pumping may be infrequent and the total amount of energy involved comparatively small. Moreover, the cost per unit of energy for pumping supplied at off-peak hours will be less than the value per unit of the stored energy if resold in peak hours.

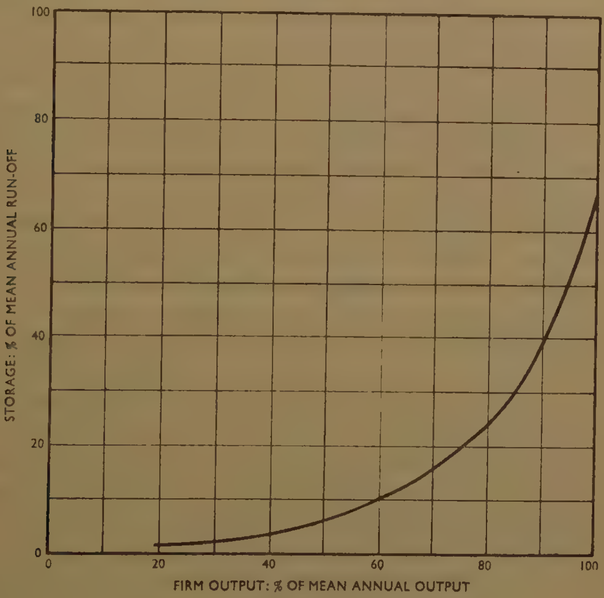

Fig. 6.—Reservoir storage curve

As an example of this use of storage pumps, assume that the mean annual output of a hydro-electric project is 200,000 MWh and that storage representing 12·5% of the mean annual output can be economically provided, but that site conditions are such that it is impracticable for this amount of storage to be exceeded. Fig. 6 shows that the firm output would be 63% of the mean annual output, i.e., 126,000 MWh. If it is assumed that the particular installation is to be designed for an annual load-factor of 24%, the firm capacity which could be installed would be 60 MW. However, if the firm output could be increased to 75%, i.e., to 150,000 MWh, the capacity which could be regarded as firm for the same annual load-factor could be increased to 70 MW.

From Fig. 6 the storage required for this increased firm output would be about 20%, and water equivalent to 7·5% of the mean annual output would have to be returned to the reservoir in the driest years. Although such years may be infrequent, additional pumping will be inevitable owing to inability to predict the weather. The cost of pumping may nevertheless be unimportant, because pumping losses are likely to be largely (or wholly) balanced by the difference in value of the off-peak and the on-peak energy. The capacity of the pumps will depend on the deficiency of natural storage and on the number of off-peak hours during which energy for pumping can be supplied at a favourable cost. Once the required pumping capacity has been derived, the annual charges on the estimated extra cost of the installation can be compared with the additional benefit obtainable from the increase of firm power and energy.

An advantage of this use of storage pumps is that the level in the reservoir can be maintained higher during periods of low run-off than it would otherwise be and the falling off of capacity due to decreased head can be reduced. This is an important advantage, and in addition the flexibility of reservoir operation is enhanced.

The reversible pump-turbine referred to in Section 6.3 is ideally suited to such a purpose. The cost of the machine is greater than that of a conventional turbo-generator, but the considerable extra cost of a separate motor-driven pump, penstock and building can be avoided. The sacrifice of efficiency of generation which is to be expected may well be acceptable on one machine in a multi-unit station in exchange for the benefit obtained. Considerable research on this type of machine has been undertaken in recent years and some notable installations are now planned for construction. The machine for Hiwassee Dam (see Section 6.3) is an example; this machine is in lieu of a conventional unit and will be erected in a spare bay constructed when the power station was built.

The Authors regard this use of pumps as an important development and think that in the planning of any storage project where there is a tailpond the possibility of having one or more pumps or reversible machines should always be considered.

(3) PUMPED STORAGE

If nature does not provide the conditions for low-load-factor hydro-electric developments, it may be possible to manufacture them, to produce a pumped storage scheme. This, in the day-time, has all the features already described of a low-load-factor scheme and the same advantages and disadvantages in its impact on thermal generation. The difference is that, during the night and perhaps at weekends, pumps have to be operated to put back from a tailpond the water which has been used in the generating periods. Pure pumped storage can therefore be regarded as the ultimate development of firming by pumping as described in the previous Section.

In a predominantly hydro-electric system there may be low-head run-of-river stations where water cannot at certain times be used or stored and hence must be regarded as surplus. Under such conditions the incremental cost of supplying power for pumping will simply be the extra running costs of the generating plant and the similar costs on the storage plant, which will be very small. The comparison is therefore between the fixed annual charges of the storage scheme and the fixed charges plus the running cost of any alternative method of generation of equal output.

If hydro-electric plant does not predominate on the system there may be no water power which can be regarded as surplus, since by correct operation it should be possible to absorb secondary energy into the system with a saving of fuel at the thermal stations. In such a case, which is the more usual, the pumping energy must be supplied from the thermal stations. The overall efficiency of dual conversion of pumped-storage energy is not likely to exceed 66–68%, excluding transmission-line losses, and hence loads can be supplied only at about two-thirds of the efficiency of direct generation. It is not possible to use the most efficient thermal plants, as they are already running at their highest possible load-factor, and supply from other stations with rather lower efficiency must be assumed. If, for example, the incremental efficiency of the plant which can supply the pumping energy is 0·28, the equivalent efficiency of the storage plant will be 0·67 × 0·28, or 0·188. A thermal plant with such efficiency would be used only at a very low load-factor, and so the storage plant is necessarily placed high on the duration curve.

The pumped-storage plant and a natural low-load-factor development are similar in their effect on the overall fuel consumption of the system, except that the storage plant has the additional handicap of the cost of the pumping energy. This is simply another factor which must be taken into account in the economic study of the all-thermal system and the alternative with a storage plant included. Such a study, taken over 25 years or more, indicates that, initially, after introduction of the storage plant the overall fuel consumption will be increased, but that the increase will diminish with passage of time owing to the effect of retarded improvement of thermal-plant efficiency lessening as the load factor of the "displaced" thermal plant decreases, i.e. as it is pushed up the duration curve by still later plant. Over a long period the total of all the effects mentioned seems to be almost zero, with possibly a very small saving of fuel in the alternative using a storage plant. This being so, it is considered that, if the capital cost of a pumped storage plant is less than the cost of a new thermal plant, the former is almost certain to give an overall advantage, bearing in mind that the sinking-fund charges are less for hydro-electric than for thermal stations.

Considerable flexibility is possible in the planning of a storage scheme. If the installed capacity and the daily load-factor are predetermined and it is practicable to provide the amount of storage necessary, the daily pumping period and the capacity of the pumps can be varied to give the best results. It can either be arranged that the water used in generation each day is returned to the upper reservoir during off-peak hours of the same day, i.e., a daily storage cycle, or a weekly storage cycle can be adopted wherein the reservoir is not fully replenished each day and a cumulative lowering is accepted, the reservoir becoming empty at the end of the week's generation. It is then refilled by pumping at the week-end. The daily cycle involves longer pumping hours each night, and it may be that the required period exceeds the time for which efficient thermal plant can be used to supply the pumping energy. If so, the pumping cost will be higher because less efficient thermal plant will have to be used. The pumping period can, of course, be reduced by increasing the pump capacity. A weekly storage cycle enables the pump capacity and daily pumping period to be reduced, but it necessitates a larger reservoir. The three variables of pump capacity, pumping period and reservoir capacity, i.e., size of dam, must be considered if the most economical scheme is to be achieved. An example is given in Section 11 ; it is hypothetical, but is based on estimates made for a storage project in Great Britain.

The Ontario Hydro-Electric Power Commission are constructing a large storage project supplied from surplus water power in conjunction with the new Sir Adam Beck Power Station on the Niagara River below the Falls. Under an agreement between Canada and the United States the balance of the water over and above that passing over the Falls is divided almost equally between them. The quantity which is to be allowed to pass over the Falls is greater in the summer daylight hours than at nights, for tourist reasons, and thus a greater quantity is available for power during summer nights than in the day-time. The project will utilize the surplus water at nights in the new generating station to provide power for six reversible pump-turbines. The main station is supplied by tunnel and open canal, and the pump-turbines, installed in a separate station, will pump from this headrace canal into an artificial reservoir. The same machines will pass the water back into the canal and produce about 220 MW during peak-load periods. This water will be used a second time in the main station, where an additional 300 MW will be installed for peak-load purposes only, making a total of 1,200 MW.

(4) NUCLEAR AND HYDRO-ELECTRIC POWER

The present intense interest in the development of nuclear power naturally raises the question of the probable effect of nuclear power stations on future hydro-electric projects. It is generally agreed that nuclear power stations are not likely to become commercially significant for a good many years ; on the other hand, large hydro-electric projects have to be planned many years ahead and will have an assumed life (for amortization purposes) of more than half a century. Thus, even at the present time and despite lack of experience in the construction and operation of nuclear power stations, it has become necessary to consider their probable effect on proposed hydro-electric schemes.

It is expected that the first of the British experimental nuclear power stations will be in operation within two years, and it is significant that the anticipated overall cost of generation is of the order of 1d. per kilowatt-hour, which may well be regarded as a surprisingly low figure for the first experiment in a totally new field. One of the principal characteristics of present nuclear fission research is the embarrassingly large number of possible designs of nuclear reactors. The fact that there are so many possibilities is a fairly clear indication that the cost of nuclear generation is likely to fall substantially with the acquisition of greater knowledge and experience.

While it is not possible to assess the cost of nuclear generation in, say, 10 or 20 years' time, it is worth considering certain probable characteristics of such stations :

(*a*) For safety reasons, nuclear stations are likely to be placed in remote districts and will therefore require long-distance transmission. In this respect they will be similar to most hydro-electric stations.

(*b*) Nuclear power stations are likely to involve relatively high capital costs and relatively low running costs, and again there is some similarity with hydro-electric plants.

(*c*) For technical reasons it will probably be desirable to keep the loading of nuclear power stations comparatively constant. They are not likely, therefore, to be suited for peak-load operation and will not have the flexibility in operation which is characteristic of hydro-electric stations.

Any form of generation which has high capital costs and low running costs, but is capable of continuous operation, will obviously be most economic if operated as nearly as possible at 100% load factor. For this reason nuclear stations are likely to be operated for base-load purposes and will become less economic if and when all base-load has been supplied and further development might therefore have to be at lower load factors.

At this stage it may possibly pay to convert the base-load characteristics of nuclear power stations into peak-load characteristics by means of pumped storage, either in the form of pure pumped-storage schemes or in the form of low-load-factor hydro-electric projects augmented by the provision of pumping equipment.

This is looking far into the future, however, and the immediate question is whether the prospects of cheap nuclear energy might make it advisable to postpone hydro-electric development and, in place, build shorter-life thermal stations. The argument would be that such thermal stations would involve less immediate capital expenditure and would be nearing the end of their lives by the time nuclear power stations were fully commercial.

Such a policy would, however, involve higher total generating costs initially, assuming that the postponed hydro-electric schemes were attractive in comparison with thermal generation.

On the basis of present knowledge such a policy would seem imprudent. In the first place, expectations regarding nuclear stations may not be realized for considerably longer than at present anticipated. Even on the most optimistic assumptions it is likely to be at least a decade before the costs of nuclear energy fall below those of coal-fired energy, and it may be still longer before nuclear stations compete with hydro-electric stations. In the second place, there is the ever-increasing necessity in this and other countries to husband limited coal resources. The prospect of base-load nuclear energy cheap in running cost does, however, tend to strengthen the arguments in favour of relatively low-load-factor development of hydro-electric stations. It may well be that such stations will prove complementary to future nuclear stations.

It is tempting to speculate as to whether some of the techniques of hydro-electric construction, particularly underground construction, will eventually prove of value for those types of high-speed breeder reactors which are not inherently safe. It would appear that safety might be achieved by deep excavation in suitable rock and that economy might also be obtained by use of natural rock in place of concrete for biological shields and other purposes.

(5) THE EFFECT OF RISING PRICES

There is a further factor which affects all hydro-electric development. During the past half-century there has been a fourfold increase in prices and a corresponding reduction in the value of money. There is a very general expectation that prices will continue to rise, and the view has even been expressed that gently rising world prices are desirable to ensure full employment. It certainly seems probable that the trend of the past half-century (and longer) will continue into the distant future, and it is of importance to consider what such an assumption implies in regard to investment in generating plant.

In order to illustrate the effects on thermal and hydro-electric plant investment, Fig. 7 has been drawn on the assumption that prices will rise during the next half century or so at an average rate equal to that of the past half century. Two alternatives are considered :

(*a*) Investment in a thermal station whose capital cost is £x.

(*b*) Investment in a hydro-electric station of similar installed capacity whose capital cost is £$2x$.

Both investments are assumed to be equally profitable initially, and the thermal station is assumed to have a life of 30 years and the hydro-electric station a life of at least 60 years. It is further assumed that construction is financed by loan capital repayable at par.

At any moment after year 0 the benefit in respect of capital investment will be related to the difference between the original capital cost and the capital cost of building an equivalent station at the date in question. The area marked A, which is the integration of the difference between the original and the replacement cost multiplied by time, can be regarded as being roughly proportional to the total benefit in respect of capital investment accruing to the owner of the hydro-electric station as a result of falling value of money. Areas B and C represent the corresponding capital investment benefit to the owner of the thermal station. The large excess of area A over areas B + C is indicative of the benefit which accrues to the equity owner of a large long-term capital investment such as a hydro-electric project. It must, however, be stressed that Fig. 7 is based on a number of assumptions and ignores various effects such as changes in interest rates and changes in replacement costs due to technological advance. It is therefore merely illustrative of the general effect on capital investment of rising prices.

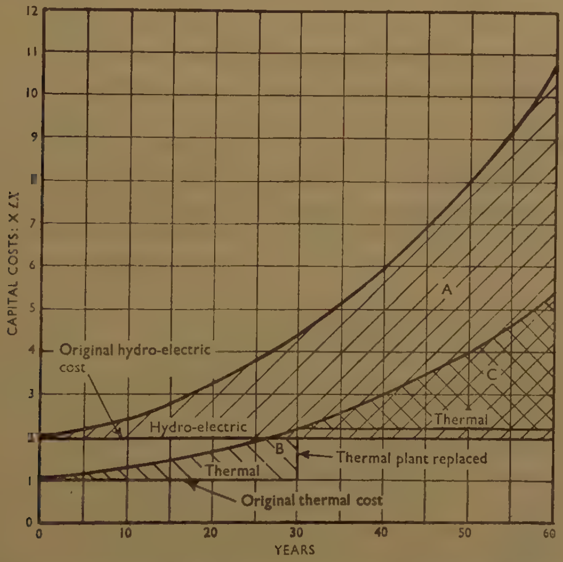

FIG. 7.—COMPARISON OF RELATIVE PRICE LEVELS AND REPLACEMENT COSTS

From a national point of view, on the assumptions made, the gain which accrues to the owners of the plant will tend to be balanced by the losses incurred by those who lend money repayable at par, so that it would be false to draw the conclusion that long-term capital investment is necessarily in the national interest. Nevertheless, the relatively large benefit to the equity owners of long-life hydro-electric stations as compared with the benefit to the equity owners of the shorter-life thermal stations is striking—particularly when it is borne in mind that a large part of a hydro-electric station, e.g., most of the civil-engineering works, will not have to be replaced even at the end of 60 years. As examples of this benefit mention can be made of the two Grampian Power Company stations (built about 1933) and the Galloway stations (built about 1936). The original capital costs of these works are now much lower than present-day costs, and the stations consequently contribute to keeping down the cost of electricity.

(6) HYDRO-ELECTRIC PLANT

(6.1) *Water turbines*

There has been no significant change in the fundamental design of hydraulic turbines for general power development since the introduction of the movable-blade Kaplan machine more than 30 years ago. Development has been concentrated on improvement of efficiency, greater output, and higher specific speed and upper limit of head for each type of turbine.

The present levels of turbine efficiency are about 93% for Kaplan, 92% for Francis and between 90 and 91% for Pelton turbines. These figures can only be a general guide, because the maximum efficiency obtainable with any type depends on the specific speed of the runner. Higher figures have been published, but it may well be that these have been influenced by the inherent inaccuracy of the various methods of measuring water flow.

Reaction turbines still suffer, to a degree varying with the type and specific speed, from a drop of efficiency at maximum and at partial loads due to the use of guide vanes to regulate the flow of water into the runner. Experiments being carried out with a type of guide apparatus which will produce annual flow at the runner inlet could give rise to an important development, since by this means the present disadvantage of many moving parts and poor part-load efficiencies would be alleviated.

Turbine outputs have been gradually stepped up to the existing limits of 165,000 b.h.p. for Francis machines at the Grand Coulee power station and 111,000 b.h.p. for Kaplans at the McNary Dam. A contract for Francis turbines of 175,000 b.h.p. of British design has recently been placed by the Quebec Hydro-Electric Commission for the Bersimis station. Single-jet twin runner Pelton turbines of 150,000 b.h.p. are now being manufactured for Cimego power station in the Italian Alps, and 4-jet single-runner vertical impulse machines of about the same output are being installed in the Kemano station in British Columbia. This represents a considerable advance in output of impulse turbines. The continuance of this upward trend in capacity is a matter more of economics than of any technical or design problem. Although the cost per horse-power of the whole machine will normally be reduced as the capacity increases, there is a limit beyond which the reverse will result and which can only be determined in each particular case. The optimum capacity is affected by problems of transport to site and erection and to some extent by the particular manufacturer's production facilities. It is probable that there will not be many future installations where such large-capacity machines as those mentioned are economically or operationally suitable, and it is unlikely that there will be much increase above these outputs.

The desire to reduce cost leads to higher speeds being adopted for a given combination of head and output. This results in the upper limits of head for Kaplan and Francis turbines being continually increased, so that the higher-specific-speed Kaplan turbine is used up to heads which hitherto had been considered technically practicable only for the Francis type; similarly, the Francis turbine encroaches into the high-head range of the Pelton. The highest heads for which Kaplan and Francis machines have yet been constructed are respectively 230 ft at the Bort-Rhue plant in France and 1,490 ft at Fionnay in Switzerland. The upper limit of head for a pelton wheel is determined solely by the ability to obtain adequate strength of buckets and wheels; the highest existing head at present used is 5,800 ft at Reisseck in Austria, and particulars of some of the highest-head plants of each type are given in Table 2.

TABLE 2.—SOME EXISTING HIGH-HEAD TURBINES

Station	Country	Head	Output	Speed
		ft	b.h.p.	r.p.m.
Kaplan Turbines				
Bort-Rhue	France	230	31,500	375
Barcis	Italy	199	13,500	500
Requejada	Spain	188	5,700	500
Invergarry*	Scotland	177	28,000	250
Pollaphuca	Ireland	165	25,000	300
Ligga	Sweden	130	105,500	125
Lavey	Switzerland	125	30,000	214
Francis Turbines				
Fionnay	Switzerland	1,490	63,000	750
Limberg	Austria	1,430	77,500	500
Vinstra	Norway	1,360	69,000	500
Fiastrone	Italy	1,320	20,000	1,000
Lages	Brazil	1,100	54,000	600
Lardit	France	1,100	30,000	750
Pelton Turbines				
Reisseck	Austria	5,800	31,000	750
Dixence	Switzerland	5,700	50,000	500
Mieville	Switzerland	4,750	47,500	500
Pragnieres	France	3,920	100,000	428

* Under construction.

These advances have been made possible by the use of materials which have better resistance to cavitation and corrosion and also by the considerable research which is carried out in hydraulic laboratories and testing stations into the problem of cavitation and runner blade profiles. The use of stainless steel for all parts of a turbine subject to erosion can frequently justify the extra cost compared with plain carbon steel by saving outages for repair, particularly with medium- and high-head machines, and a stainless-steel runner can enable the depth of a machine relative to the tailwater level to be reduced, thus saving in cost of excavation of the foundations. A material containing 12–14% of chromium with not more than 1% of nickel is now favoured. Manganese bronze has been much used for turbine runners, particularly those of small dimensions, but aluminium bronze containing about 10% of aluminium and 1% of iron has even better resistance to cavitation ; moreover, it has good tensile strength and can be welded.

In the choice of the most suitable speed for the machines for a particular installation—and the selection of the type of turbine if the conditions are within the overlapping range of two types—it is important to realize that the highest speed will not necessarily give the lowest overall cost of the installation. An increase of specific speed is usually accompanied by a lower maximum efficiency and a greater depth of excavation necessary for the draft tube ; this is particularly so with Kaplan turbines. Thus the choice must be made with full knowledge of the incremental cost of excavation for the foundations and of the value of efficiency. Maximum efficiency may not be the main consideration, and weighted efficiency over the anticipated operating range of output is usually the more important criterion.

In recent years there has been a greatly increased use of steel plate and fabricated construction, which affords reduction in weight and therefore in cost, and welding has almost completely superseded riveting. Some manufacturers prefer to fabricate Francis runners whenever this is practicable, and thereby avoid the risk which always arises of castings being faulty. The welding of pressed-steel blades to a cast or rolled rim and a cast hub permits pre-machining of the rim and hub to smooth and accurate contours and gives a blade surface which requires little hand grinding. A fabricated Francis runner, however, is not necessarily cheaper than one of cast steel ; this depends on the ability of the steel founder to produce a sound and accurate casting which requires the minimum of cutting, welding, and grinding of flaws.

It is unlikely that there will be any major change in mechanical governor design in the future, although improvements in detail will doubtless be made. Extensive interconnexion of power-supply systems and individual plants has changed the function of a turbine governor from that of pure speed control on an isolated system to that of a frequency/power regulator on the interconnected system. It must often be capable of control by any of several quantities, e.g., by frequency and output, water level, water flow or power flow in the interconnexions, and it must be possible to mix these quantities and vary their relative magnitudes. In this respect the new electronic governor which has been developed in Sweden shows considerable promise. This embodies an electronic-valve regulator to operate the relay valve of the turbine servo-motor instead of the normal pendulum-operated linkage used in the purely mechanical governor. Its principle is the oscillating circuit comprising two balanced electronic valves in push-pull, a resonant circuit responding to frequency which is supplied from a tachometer-generator or voltage transformers on the turbo-generator, and an RC circuit to provide the damping and feedback which is supplied from a potentiometer operated mechanically from the turbine guide-vanes. The quantities mentioned above are all introduced electrically by potentiometers.

The common methods of measurement of water flow for the purpose of establishing turbine efficiency have an inherent inaccuracy, and an appreciable tolerance, usually accepted as 2%, is normally required by the manufacturer. Measurement at site involves an appreciable expenditure on equipment and outage of the machine being tested. Furthermore, tests can be taken only at the head available at the time, and if the machines are designed to operate over a large range of head the information obtained is not as complete as the owner usually requires. It is often necessary, therefore, to accept the machine the basis of tests at one head and to use the manufacturer's design data for other conditions of head and output.

Because of this insufficiency, there has been an increasing use of reduced-scale models tested under laboratory conditions; such models must be complete and entirely homologous with the prototype, and the scale will be such that the runner diameter is of the order of 500–650 mm. Complete information and reliable results can be obtained and the design can be modified in the light of model tests if these are completed at an early date. There is, however, doubt as to the accuracy of any of the usual scale-effect formulae used in calculating the absolute value of the prototype efficiency. The formulae have no exact theoretical basis and are derived principally from experiment on full-scale machines and homologous models. They can be interpreted in different ways, particularly their application to efficiencies at loads above and below the optimum, and it is important that the particular formula and its method of application are agreed between the purchaser and the manufacturer, preferably before a contract is placed.

Whether or not this method gives the true prototype efficiency over the whole range of tests, it does produce results which from the contractual point of view cannot reasonably be disputed and no tolerance need be granted. This may be important if a large bonus or penalty on guaranteed efficiencies has been agreed in the contract. Turbines can be accepted on the basis of guaranteed model efficiencies, thereby removing, from the contractual aspect, any doubt about the scale-effect formula. A complete model may cost as much as £5,000 to manufacture and test, but such expenditure may well be justified. However, unless manufacturers can have the opportunity of tests at site and on complete models, it will probably not be possible further to determine the most satisfactory scale-effect formula to be applied to the model results. Even so, the disparity between the results obtained by using different formulae is almost certainly less than the limits of the tolerance accepted on site-test results. An indication of this is given in Fig. 8; the curves show actual model test

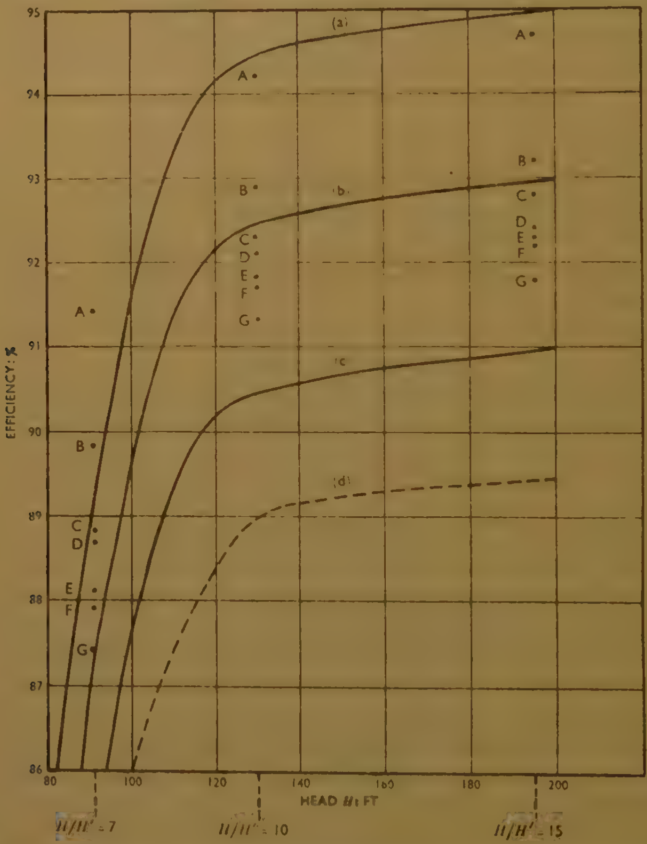

FIG. 8.—MODEL TEST AND PROTOTYPE EFFICIENCIES DERIVED FROM DIFFERENT SCALE-EFFECT FORMULAE

Model scale ratio = 5·23; model test head $H' = 13$ ft.

(*a*) Upper tolerance limit for site tests.
(*b*) Guaranteed.
(*c*) Lower tolerance limit for site tests.
(*d*) Model test results.

efficiencies at various prototype heads, together with the guaranteed prototype figures and the limits of a ± 2% tolerance for site testing. The points numbered A–G, which are shown for three ratios of prototype head to model head, are the prototype efficiencies derived from the model results and seven of the various formulae which have been produced. If point A, which is exceptionally high, is disregarded, it appears that in this particular example the remaining six points fall within a range of efficiency of about $1\frac{1}{2}$–$2\frac{1}{2}$% (absolute). It must be stressed, however, that the figures given relate only to optimum efficiency at each head; the curves do not take into account efficiencies at part-load or overload.

(6.2) *Generators*

Developments in design of vertical-shaft generators have been fully described in recent papers.[12, 13] The tendency for turbine outputs and speeds to be increased naturally reflects on the generator. With low-speed machines, having a rotor peripheral speed at normal running below about 15,000 ft/min, the problem of increased output is principally one of transport to site and erection. Generators having rotor diameters between 30 and 40 ft and running at only 75 r.p.m. exist, and it is usual for such machines to be taken to the site in pieces as large as can conveniently be transported, and then erected at site for the first time. This means that certain tests which it is practicable to carry out only in the manufacturer's works cannot be made. Provided that the generators are designed for initial erection at the site, there should be no difficulty with low-speed machines of providing such outputs as are likely to be required in the future.

With high-speed generators the problem is more that of the mechanical design of the rotor poles and coils to withstand the centrifugal stresses due to increased runaway speeds which are entailed by higher turbine specific speeds. However, the large outputs of high-speed machines recently built are well below what is attainable; the peak occurs in the speed range 400–600 r.p.m. and exceeds 200 MVA.

The demand for an increased rate of exciter response and restriction of voltage rise on load rejection is leading to the replacement of the rheostatic excitation-control system by one of an electronic type from which much closer voltage regulation is obtained. Systems have been developed which are fundamentally similar in that the normal vibrating-contact or sector-type automatic voltage regulator is superseded by a magnetic amplifier and the pilot exciter is omitted.

(6.3) *Reversible pump-turbines*

Mention has been made earlier of the reversible pump-turbine. A reversible machine of the propeller type was installed in Germany about 20 years ago, but as such a machine is restricted to operation with a head within the range of a Kaplan turbine, its general use in pumped storage schemes is limited. Considerable research and model tests have been carried out in recent years on reversible machines with runners of the Francis type which could be used up to much higher heads. Such a machine must have a runner design which is a compromise between that best for a pump and that best for a turbine, since one speed cannot be the most suitable for both operations. The machine will be larger than a conventional Francis turbine of the same specific speed but smaller than a normal pump. A notable machine of this type will be installed at Hiwassee Dam of the Tennessee Valley Authority. According to published information, it is designed to deliver 3,900 cusec against a head of 205 ft when operating as a pump and requires an input of 102,000 b.h.p. Nominal rating when operating as a turbine is 80,000 b.h.p. at 190 ft head.

It is likely that there will be an appreciable demand for a reversible machine, and there would seem to be scope for further technical advance. An alternative to the compromise runner design is to have a 2-speed electrical machine, one speed for generation and a higher speed for pumping. For low heads a further alternative is a Kaplan-type runner with blades which could be reversed for pumping without changing the direction of rotation of the machine.

(6.4) *Control of plant*

The control of hydro-electric plant was the subject of a paper[14] presented to the Institution of Electrical Engineers a few years ago. The alternative methods can be classified as fully manual, semi-automatic and fully-automatic control. Opinions of operating authorities vary on the relative merits of these methods. Some do not favour semi-automatic control, because trained staff are required in the turbine room and it is thought desirable that they should be more fully occupied with the machines than is required by push-button semi-automatic operation; others consider that fully automatic control is a complication and wish to avoid the extra maintenance of a specialist nature which it involves.

From the operational aspect, fully or semi-automatic control enables machines to be started in the shortest possible time, and thus the inherent advantage of hydro-electric over thermal plant in a combined system (of rapid starting and stopping) can be fully utilized. From the economic aspect, the extra cost and maintenance of the control equipment and supervisory cables has to be compared with the cost of the salaries and overheads of such staff as can be saved. In particular, for small and isolated stations it is to be expected that fully automatic control will show an appreciable saving.

It is probable that large base-load plants will continue to be staffed and be either manually or semi-automatically controlled, but it seems likely that the use of fully automatic remote control for smaller-peak-load plants will be extended, especially when they are situated in lonely and intractable country.

(7) TRANSMISSION AND INTERCONNECTION

Good hydro-electric sites are frequently remote from large load centres, so that the electrical transmission of their output over long distances is often a necessity. In the past, technical inability to transmit over the distances involved has been a deterrent, but recent development of transmission technique has brought within reach more and more of the cheap but remote sources of water power throughout the world. In many cases their exploitation is being speeded by mounting prices or shortages of fuel.

From the aspect of deciding how a given water-power scheme should be developed, or whether it should be postponed in favour of some alternative, the costs of the scheme itself and the necessary transmission are inseparable and must be the basis of economic studies such as are referred to in Section 2. Large hydro-electric schemes, particularly if remote, may call for very high voltages and the consequent use of multiple or " bundled " conductors to reduce corona loss and reactance, but there are a number of other methods whereby the load capacity of long lines may be increased. These are mainly directed towards compensation of the line reactance and capacitance and the maintenance of transient stability, and include the following :

(*a*) High-speed switching.
(*b*) Intermediate switching stations in double or multiple-circuit lines.
(*c*) Rapid control of generator excitation.
(*d*) Shunt compensating devices in the form of synchronous condensers or reactors.
(*e*) Series capacitors.

All of these methods have been recognized for many years, at least in theory, but it is only comparatively recently that the need for their co-ordinated use has resulted in the technical development work, both theoretical and practical, being carried out.

The collective effect of all the foregoing methods of improving A.C. transmission is so considerable that, for the present at least, there is no great urgency to develop D.C. transmission in those areas where A.C. overhead lines can be constructed. D.C. transmission offers the possibility of transmitting very large blocks of power over distances beyond the likely capacity of A.C. transmission, even at much higher voltages than at present in use. Such transmission may therefore become of importance in the future, although the prospect of this for overhead routes now seems more remote than at one time was thought probable. It seems likely that D.C. transmission will in the first instance be developed for submarine or underground routes, where its advantages are so outstanding. One such case is the proposed link between Great Britain and the Continent [18] which, although likely to be an A.C. system initially, may later be greatly increased, and possibly cheapened, by the use of D.C. cables.

The selection of the best transmission system for any given hydro-electric project or group of projects involves such fundamental assumptions as future fuel prices, life of equipment, rates of load growth, probabilities of outage, etc. Some of these may be little more than guesses, so that it is not usually possible to arrive at one clear answer by calculation. A careful study, however, enables definitely unsuitable alternatives to be eliminated, leaving the final choice to be based on judgement and experience.

Within the boundaries of a sovereign State, adequate transmission and interconnexion of capacity can be provided to achieve full integration of hydro-electric and thermal plant, and the benefits of such full integration can be very considerable. There is, however, the further possibility of interconnecting the systems of two or more adjacent countries, thus reducing the required amount of plant. This interconnection is likely to be particularly advantageous if the two or more countries in question vary considerably in characteristics. Such circumstances arise where one country has predominantly thermal generation and the other predominantly hydro-electric generation ; but even if both have predominantly hydro-electric generation there may be important diversities arising from the differing characteristics of the stations. For instance, one country may depend upon winter rainfall, while another may depend upon the melting snows in spring and early summer.

The maximum benefit would be obtained if such countries were fully integrated in the manner possible within the boundaries of a single State, but for political reasons it is usually impossible for one sovereign State to be dependent upon another for vital supplies of electricity. Partial integration by means of high-voltage interconnection can nevertheless be of great advantage, even if the actual net interchange of energy is arranged to be approximately zero over a complete year. There has, in fact, been a rapid increase in the number of high-voltage transmission lines connecting one European country with another. In the particular case of France and Great Britain, recent investigations have shown that interconnexion could result in a saving of plant amounting to some 300 MW.[18] This saving may more than offset the whole capital cost of interconnection, but political considerations may prevent its full exploitation, bearing in mind the criterion already mentioned that one State cannot afford to be wholly dependent on another. In other words, the reduction of installed plant in the two interconnected countries must not be so great as to leave either country serious embarrassed if the interconnection is severed.

(8) CONCLUSION

The paper has been more concerned with drawing attention to the problems (and possibilities) arising in combined thermal and hydro-electric systems than with solving these problems. To many of them there is no certain solution, mainly because of inherent lack of adequate data, particularly those relating to the future. This, however, merely means that maximum use must be made of such data as are available and that the problems must be approached with full knowledge of both thermal and hydro-electric operating characteristics. Difference of opinion between thermal and hydro-electric engineers as regards the design of projects is not uncommon and may arise because of inadequate understanding by the one of the problems of the other, or lack of mutual appreciation of the inherent characteristics of the two types of plant.

The starting point in the design of a hydro-electric project is the hydrographical and civil-engineering data, but the final design must depend on the result of a study of the other problems and possibilities relating to combined thermal and hydro-electric operation. This is especially true where, for instance, there are possibilities of pumping or making provision for future increase in installed capacity, or where an exceptionally long transmission is involved. Consideration of the latter is necessary from the outset, since transmission cannot be treated as an independent problem.

Even on the assumption of the fullest use of all available data, and the most careful study of the various interrelated problems, it remains probable that the best which can be achieved is to reduce to narrow limits the range of uncertainty regarding the principal hydro-electric design features. The final decision within this range will then be a matter of judgement. If a particular project turns out to be marginal, or barely economic, it will be necessary to decide whether it is justified as a fuel-saving measure, or whether the probability of continued rise in prices and fall in money values should be taken into account. The effect of this factor can well convert a present marginal project into a highly profitable project in the future.

The technical development of hydro-electric plant has reached a stage where there is limited scope for further improvement in design and efficiency. Since the cost of civil-engineering works predominates in the overall costs of most hydro-electric installations, it is mainly in this field that improvements in design and construction methods can be looked to for any appreciable reduction in the overall cost of water power. Such savings as may be achieved in this respect in the future may well be offset by the need to develop power at sites which are increasingly less favourable. On the other hand, a continuation of the steady improvement in thermal-plant efficiency is likely for some years to come, but this may be more than counterbalanced by further increases in the cost of fuel.

The initial cost of thermal power can be estimated in advance within close limits and is largely independent of the location of the plant. The future cost depends on the price of fuel. The capital cost of hydro-electric installations must be estimated from a detailed investigation of each individual site and is not necessarily related to that of any other similar plant. Even after a careful survey, unexpected difficulties encountered with the civil-engineering works may arise and cause the original estimates to be exceeded. Thus, the cost of thermal power can be closely estimated at the time the plant is built, but is liable to unpredictable future variations. The cost of water power, whilst more difficult to assess in the first instance, remains virtually constant once the plant has been constructed.

(9) ACKNOWLEDGEMENTS

The Authors would like to acknowledge the invaluable assistance received from their colleagues in the preparation of the Paper. They also wish to express thanks for data made available to them by various persons.

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(11) APPENDIX

COST ESTIMATES OF A PUMPED-STORAGE SCHEME

The example which follows serves to illustrate the flexibility which is possible in the planning of a storage project. The scheme envisages a single pumping stage between a large natural lake and an artificial reservoir in the surrounding hills. Approximate calculations and estimates are given in Table 3 for an installed generating capacity of 200 MW, and for three alternative proposals, (*a*), (*b*) and (*c*). Alternative (*a*) is based on a weekly storage cycle, with pumping restricted to 7 hours each night, whilst (*b*) and (*c*) cover a daily storage system, (*c*) having the shorter pumping period compensated by additional pump capacity. Both (*b*) and (*c*) have only 70% of the reservoir capacity of (*a*).

TABLE 3.—ESTIMATED DATA FOR THREE PUMPED-STORAGE SCHEMES

	Scheme (*a*)	Scheme (*b*)	Scheme (*c*)
General Data			
Daily pumping period, hours	7	8	7
Type of storage	Weekly	Daily	Daily
Generating capacity, MW	200	200	200
Pumping capacity, b.h.p. $\times 10^3$	268	268	301
Storage required, for daily generation, ft$^3 \times 10^6$	91	91	91
Storage obtainable each day (full-load pumping), ft$^3 \times 10^6$	81	92	91
Deficiency each day, ft$^3 \times 10^6$	10	Nil	Nil
Extra pumping at weekend (Saturday and Sunday), hours	4½	Nil	Nil
Reservoir working capacity required, ft$^3 \times 10^6$	131	91	91
Energy			
Generated annually, MWh $\times 10^3$	286	286	286
Required for pumping, MWh $\times 10^3$	432	432	432
Delivered to point of supply, MWh $\times 10^3$	275	275	275
Losses, MWh $\times 10^3$	157	157	157
Power			
Delivered (less transmission losses), MW	193	193	193
Costs			
Capital (including transmission), £ $\times 10^3$	8,400	8,200	8,650
Annual:			
Capital charges, £ $\times 10^3$	404	396	420
Operation, etc., £ $\times 10^3$	20	20	20
Cost of pumping energy, £ $\times 10^3$	810	828	810
Total, £ $\times 10^3$	1,234	1,244	1,250
Revenue			
Sale of power, £ $\times 10^3$	772	772	772
Sale of energy, £ $\times 10^3$	745	745	745
Total	1,517	1,517	1,517
Annual surplus, £ $\times 10^3$	283	273	267

The following assumptions are made:

Mean generating head	600 ft
Mean pumping head	636 ft
Generating period	5½ hours per day, 5 days per week
Daily load factor	23%
Annual load factor	16%
Overall efficiency of conversion (including transformers)	67%
Transformer losses	2%
Overhead line losses	1½%
Interest on capital and on sinking fund	4%
Value of power	£4 per kW
Value of energy, on-peak	0·65d. per kWh
Value of energy, off-peak (for 7 hours)	0·45d. per kWh
Value of energy, off-peak (for 8 hours)	0·46d. per kWh

It will be seen that, in this particular analysis, alternative (*a*) appears the most favourable, but it is evident that the annual surplus is very sensitive to the assumed cost of energy for pumping. Thus the pumping period should be the longest each night for which the energy can be obtained at the lowest rate and there will be some period which it is not economic to exceed.

It may be that the plant will not be required to generate each day the maximum energy for which it is designed (as assumed in Table 3), in which case the annual surplus becomes greater, since, provided that the kilowatt earning capacity is always maintained, every kilowatt-hour generated represents, in this particular example, a definite financial loss. If, for example, only three-quarters of the maximum annual energy is produced, the annual surplus for alternative (*a*) becomes £299,000. It may be considered proper to allocate a proportion of the kilowatt capacity to the general system allowance for breakdown, in which case credit cannot be taken for the full installed capacity. If such allowance were 10% the revenue from sale of power, and hence the annual surplus, would be reduced by about £80,000.

The load factors in this particular example are higher than is often possible for pumped-storage schemes, and if lower load-factors were permissible the annual surplus would be greater.

The Paper, which was first received on the 21st May, and in revised form on the 3rd August, 1954, was presented to the Institution of Electrical Engineers.

Discussion *

Mr T. Lawrie: It is now over 26 years since Mr Haldane's firm first advised on hydro-electric schemes with which I was concerned in the Highlands, and after many vicissitudes and lost opportunities there is now in the north of Scotland a large and rapidly-growing development of hydro-electric power. The outstanding advantage of this power is that it is distributed at reasonable prices to the sparse and scattered population of an important food-producing area and is exported as peak-load power to central Scotland. Perhaps its most dramatic advantage is that it is saving 700,000 tons of coal per annum, or nearly a fortnight's production of the

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* Reprinted from Proc. Instn Elec. Engrs, vol. 102, Part A, June 1955.

Scottish coal pits. This represents a hidden dividend of 5% on the cost of the hydro-electric schemes, in addition to what they earn as economic producers of electricity. Moreover, this coal saving will be doubled within the present decade as hydro-electric schemes now building come into production.

A third great advantage from the national aspect is that here at last is a first-rate hydro-electric shop window. Manufacturers of hydro-electric plant, civil engineering contractors and consulting engineers can now take potential customers from abroad to see the many dams, power stations and tunnels which have been built and equipped here at home in Scotland, and they can demonstrate in a practical way our ability to compete with European and American concerns in all the continents of the world and even, I am glad to say, by breaking into the United States market.

This is one point which the authors might perhaps have brought out, on grounds both of technical interest and of the impression which it will create at home and abroad. Table 1 gives the approximate proportions of thermal and hydro-electric plant in various countries, and in Scotland we really have a mixed thermal and hydro-electric system, as the following facts and figures show. In the Hydro-Electric Board's Area in the North of Scotland, north of the Firth of Tay and the Firth of Clyde, there is a higher proportion of hydro-electric plant than there is even in Sweden, while in the whole of Scotland the proportion of hydro plant is 50% greater than in the United States. The figures are as follows for 1954:

	Thermal %	*Hydro-Electric* %
Hydro-Electric Board's Area .	22	78
Whole of Scotland	67	33

To take the matter even further, when hydro-electric schemes now under construction are completed towards the end of this decade, the proportions in the Hydro-Electric Board's Area will be 14% thermal and 86% hydro-electric.

I should like to describe the problems of hydro-electric design in the Hydro-Electric Board's mixed thermal and hydro-electric system and how they have worked out in practice. In the first place, Scotland's rainfall is much higher in winter than in summer, and the average monthly run-off into the Board's reservoirs follows very closely the average monthly demand for electricity. This is a great advantage, which offsets the disadvantage of our smaller catchments compared with Continental countries or our low heads of water compared with Norway and Switzerland; ample storage is nevertheless desirable, because of occasional severe freeze-ups in January and February. Secondly, the availability of hydro-electric plant is remarkably high, because power can be obtained quickly at the turn of a tap and because outages for repair and maintenance or because of breakdown are very small indeed—much less than for steam plant.

Those are the foundations on which we are operating in the North, and peak-load water power has been developed to a large extent in Scotland. It started with the Galloway scheme 25 years ago, which develops 102 MW at 20% load factor, and the Hydro-Electric Board have carried the idea further with their first large scheme at Sloy, 130 MW at 10% load factor, followed by two more schemes. Shira with 45 MW at 20% and Errochty with 75 MW at 15%, totalling some 250 MW at an average load factor of $12\frac{1}{2}$%. Some of this peak-load power is absorbed in the Board's own area and a great deal is exported to Central Scotland.

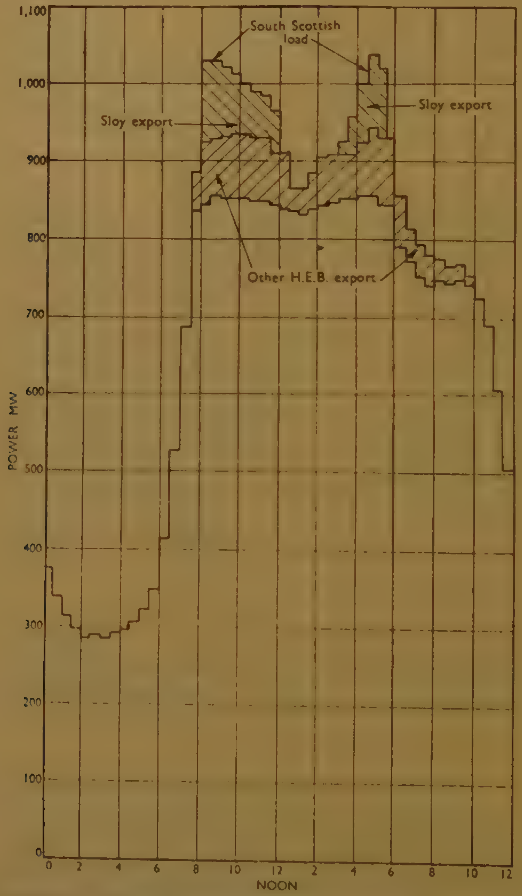

FIG. 9.—TYPICAL HYDRO-ELECTRIC BOARD EXPORT AND SOUTH SCOTTISH LOAD IN DECEMBER 1952

Fig. 9 shows, for a typical winter's day in 1952, how the extreme peak-load power exported from Sloy and the more moderate peak load from the rest of the Board's stations fit in to the total load curve of Central Scotland. The total load curve for Central Scotland is the line which goes over the top, and the peak-slicing operations are shown, first at the extreme peak and secondly at the moderate peak, enabling the steam plants to run at a constant load for roughly the eight working hours of the day. The steam plant in Central Scotland is thus enabled to operate with greater thermal efficiency at a steady load throughout the working day. In practice, the peak load exported to Central Scotland is a fixed maximum, at present 215 MW, and after allowing for this we have a further 216 MW of hydro-electric plant, mainly base load, and 121 MW of steam plant, providing the mixed system in the Board's own area, the successful combination of which is interesting. If the weather is dry and the level of water in the reservoirs is falling, steam production is increased ; if the weather is wet and reservoir levels are rising, steam production is cut down. Last year produced extreme conditions in both directions. February was exceptionally cold and dry, with no water coming into the reservoirs, and in that month 40,000 MWh of steam power were generated. October was quite the opposite, being exceptionally wet, and steam production fell to 10,000 MWh per month.

In conclusion, I should like to add a few words to the interesting remarks of the authors about pumped storage. There were pumped-storage plants on the Continent—in the Black Forest, the Ruhr and elsewhere—before the war. I was concerned in 1935–36 with an attempt to promote a 225 MW scheme at Loch Sloy for pumped storage. The site conditions were attractive, with a large low-level reservoir in the shape of Loch Lomond. When the Hydro-Electric Board took up the scheme after the war, however, it proved more economical to bring in water from the neighbouring catchments by tunnels and aqueducts and do without pumps. We are now, however, building part of the Shira scheme in Argyllshire as a reversible scheme with a fairly small installation of 5 MW of pumping capacity.

As the authors say, there may be more need in future for pumped storage on a large scale to enable the new large high-temperature high-pressure high-efficiency steam turbines to run at steady base loads, and, of course, any atomic power which may be produced ; but the criterion of the success of future pumped storage schemes is going to be whether the greater fuel economy secured by the steady operation of efficient steam plant will offset the inevitable losses in the pumping and generator cycle of a pumped-storage scheme.

Mr R. W. Mountain: In Section 1 the authors say " hence, changes in the average annual load-factor of a particular project usually mean changes in the installed capacity," while in Section 2.1 they say " the storage works are not directly dependent upon the installed capacity of the plant." The implication of these two statements taken together appears to be that the cost of storage or the amount of storage is to some extent constant over a wide range of design load-factor for a given catchment area.

Some nine years ago, J. K. Hunter and I tried to generalize,[18]—I now think wrongly—from an examination of one scheme, and it did appear that the cost of storage is bound to vary rather rapidly in any given catchment area with the design load-factor. I should like to ask the authors, therefore, whether that is so and whether it affects at all the conclusions to which they have come. They may say that part of the answer is to be found in Section 2.5, where there is a reference to the amount of storage, and in Fig. 6, which deals with the relation between firm output as a percentage of mean annual output and storage as a percentage of mean annual run-off.

In association with the Galloway scheme I should like to pay a rather belated tribute to the genius of the late Colonel McLellan, who foresaw some 25 years ago many of the problems which are discussed in the paper and have been dealt with by the North of Scotland Hydro-Electric Board and others in regard to mixed thermal and hydro-electric schemes.

Would the authors agree that one of the values of peak-load or very-low-load-factor plant is its use essentially as standby plant? The suggestion is that it might be more economical to provide standby plant by this means than by calling on the older thermal plant in other parts of the country.

Mr F. J. Lane: Figures given in the paper on the cross-Channel cable [19] indicated that the installed hydro-electric capacity of the British Electricity Authority was about 1% of the total and the energy production about 0·5%. Nevertheless the hydro-electric contribution is greatly valued and any increase would be welcome. The B.E.A. has been anxious to develop hydro-electric power as much as possible, but, as is well known, the problem is not treated on a purely economic basis and opposition from amenity interests has been almost fanatical south of the border.

I agree with the authors that the idealized load/duration curve can be misleading. In our normal forward planning, a system is adopted whereby curves of actual demand for typical days covering the four seasons of the year are utilized, and these are applied to the estimated maximum demands for future years, at the same time having regard to plant availability and the restrictions which may occur owing to transmission, inflexibility of plant and so on.

Referring to item (*c*) in Section 2.3.2, and in particular to the statement that it is difficult to restrict a peak-load thermal station to an annual load-factor of less than about 15%, I would remark that the B.E.A. have stations at the present time operating at annual load-factors of 5% or less. By 1965 it is expected that there will be stations operating at even lower annual load-factors. At the same time it will be necessary to cope with the introduction of new thermal plant operating at higher steam pressures and temperatures, so that it will be important to develop the special techniques of low-load-factor operation if maximum economy is to be obtained from the national system.

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[18] See reference 1, p. 500.

[19] See reference 17, p. 500.

A pumped-storage project has recently been proposed for North West Wales. This venture is the first of its kind in this country, and, if legislation permits, first-hand experience will be obtained in the operation of such a scheme in conjunction with thermal plant. The pumped-storage scheme is planned to have a capacity of 300 MW, and it is proposed to connect it into the 275 kV Grid. It will be used in accordance with its position in the national order of merit. During the winter it will run over the three peaks of the day, with maximum pumping at night, but during the summer it will be used only in an emergency and allow its equivalent capacity of thermal plant to go cold, thus saving considerable banking losses. The pumping energy will be provided from spare plant capacity, on the assumption that pumping can cease immediately should the spare capacity be required, and the storage plant could if necessary go over to generation in a very short period of time.

Reference is made to the load factor to be associated with nuclear-energy generation. While it may be ideal that the load factor should be kept as high as possible, conditions in this country and the expected development of the power supplies will lead us into a position where a reduced load factor will have to be accepted. For instance, from some rough figures recently estimated, although we may start with an annual load factor of some 75% on atomic energy plant in 1965 or thereabouts, by 1985 the very much increased quantity of atomic-energy plant—say some 30,000 MW—would be operating at 66·5% load factor, with some 50,000 MW of thermal plant operating at about a 20% load factor. As with the development of the larger thermal units, therefore, techniques will have to be developed for operating atomic-energy plant at lower load factors than the ideal, simply because of the load characteristics of the systems that are being supplied.

Mr J. K. Hunter: The term "firm power" which is used in the paper requires a little amplification, because there are various degrees of firmness. In a thermal power station, provided that the plant is available and the coal is there to burn, it is true to say that virtually the whole output is firm, but with hydro-electric power the position is different. Estimates of the latter are necessarily based on estimates of run-off, which in their turn depend on the accumulation of past records. As more records are accumulated these estimates become more reliable, but they never become completely reliable and they are never better than approximations to the truth.

For this reason, reservoir storage curves such as that shown in Fig. 6 are inherently related to time. At best they show the degree of firmness of the output expressed in terms of probability. If adequate records are available it is possible to prepare a family of curves showing the percentage of years in which a selected reservoir storage will provide a given firm output; but, however long the records, such estimates are always subject to some uncertainty, and it is impossible to be sure that at some future time river flow conditions will not prove to be more unfavourable than any which have been experienced in the past.

In 1948–49 Europe experienced an unprecedented shortage of precipitation, and this caused considerable embarrassment in countries which depended on hydro-electric power for the bulk of their energy requirements. Over very extensive regions the shortage of run-off was greater than anything which had hitherto been experienced or which had up to that time been anticipated. Again in 1947–48 south-eastern Canada suffered from an abnormal shortage of precipitation which had the effect of reducing by 45% the output of energy from those stations which were situated on variable-flow rivers. The shortage of water resulted in curtailing the overall output of the Commission's system by 17%—based on the estimated dependable supply. The reduction would have been very much more serious if at that time half of the generating capacity at the disposal of the Ontario Hydro-Electric Power Commission had not been based on continuous sources of supply, i.e., at that time they relied on the Niagara and St. Lawrence rivers for 50% of their energy.

As a result of this experience it was decided to provide thermal backing to the hitherto exclusively hydro-electric system, and between 1950 and 1953 the generating capacity was increased by the construction of two major thermal stations totalling 664 MW, which greatly increased the effectiveness of the hydro-electric stations on those rivers subject to variable flow. During the summer of 1953 the Commission again experienced unfavourable flow conditions; indeed the discharge of the Ottawa river was lower than any hitherto recorded, but on this occasion the Commission were able to make good the hydro-electric deficiencies by calling for additional generation from their thermal stations. The Commission have thus amply demonstrated the value of thermal power as a means of making a more effective use of existing hydro-electric installations, and since it is thought probable, on the basis of present load estimates, that by 1962 all the important hydro-electric sources in Southern Ontario will be exhausted, they will be forced more and more to thermal generation or other sources of power.

A somewhat similar position is developing, though more slowly, in Sweden where recent estimates of load growth suggest that the economically developable water power will be exhausted by about 1970. At the present time, in a year of average run-off, hydro-electric power provides about 95% of the total requirements of the country, but attention is now being given to the desirability of planning future stations to operate at smaller load-factors than have hitherto been customary in the expectation that one day they will be integrated with thermal stations. Such integration of hydro-electric and thermal power is at present very incomplete, but as it is developed it will result in more efficient use of the country's water-power resources—a considerable part of which now goes over the spillways.

Mr A. A. Fulton: While it is true that so far as cost is concerned storage works can be regarded as independent of the power tunnels and plant, the storage needed to maintain an evenly distributed flow throughout the year is more than for a plant whose greatest output coincides with the wettest part of the year.

Instead of comparing the incremental cost of a hydro-electric plant with that of a thermal station, it is possible to decide whether to increase the capacity of the former by comparing the rate of increase in annual charges for larger tunnel and plant with the rate at which the all-in unit price of the particular tariff increases with decreasing load-factor. At current costs of tunnel driving, steel pipes and plant, it is advantageous to increase plant capacity to the maximum, because, as shown in Fig. 10, the cost curve rises less steeply than does the tariff curve.

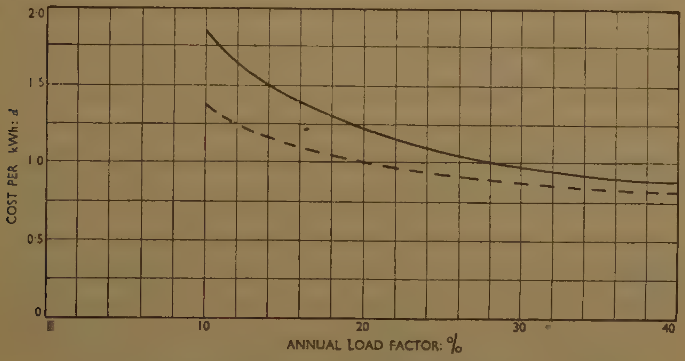

Fig. 10.—Variation of cost with annual load factor

——————— Value of energy at Grid tariff (end of 1954).

— — — — Cost of generation.

The value of coal saved is now recognized as a justification for expenditure on hydro-electricity; where an expenditure of, say, £40,000,000 results in the saving of 500,000 tons annually, the value of the coal so saved is under £4 per ton. This takes no account of the saving in capital cost of any steam plant rendered unnecessary by the provision of the hydro-electric plant.

The storage curve shown in Fig. 6 interested me particularly. I have compared it with information published in the *Proceedings of the Institution of Civil Engineers*, namely (i) W. J. E. Binnie's curve of 1931; (ii) Fig. 7 in C. M. Roberts's 1951 paper, and (iii) Table 2 of my 1952 paper. Even after adding to mine the 5% which I then recommended, I find that the authors' curve is still about 5% better than mine and fully 10% better than the others. A comparison is shown in Fig. 11. Will the authors say how they arrived at their curve?

I agree with Mr Hunter that in Fig. 6 the term "firm output" needs qualification. Its meaning is slightly different when used to describe the provision of a guaranteed even flow all the year round compared with short-term operation for, say, five days a week and at higher rates during the winter than the summer. Moreover, the evaluation of storage requires some qualification. In practice, it is customary to keep reservoir levels down to catch the odd spate and avoid spill, so the true storage can be some 5–10%, or even more, less than the nominal storage.

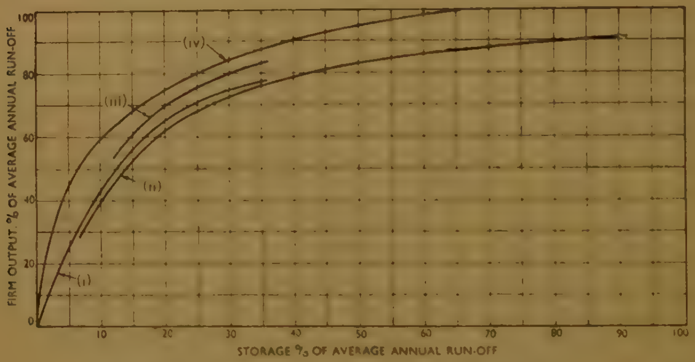

FIG. 11.—COMPARISON OF STORAGE CURVES

(i) Binnie, 1931.
(ii) Roberts, 1951.
(iii) Fulton, 1952.
(iv) Haldane and Blackstone, 1954.

The idea of special model turbine tests in replacement of full-scale site tests does not appeal to me: it is only a short step to having no tests at all. The time would soon come when the model test for one plant would be applied to another of different size, but for the same specific speed, with no more bother than an appropriate adjustment of the multiplying formula.

I do not think that there is likely to be any wholesale changeover to the new electronic governor mentioned by the authors. For large plants responsible for system frequency it would be very helpful, but for the remainder the need is rather for a governor which will keep the sets sufficiently close to their allotted share of the load to ease the job of the control engineer while ensuring the most efficient use of available water.

Mr E. M. Johnson: In Section 6.2 the authors deal with the question of generator size, and state "... the problem of increased output is principally one of transport to site and erection." The slow-speed generators to which this remark applies may be built of almost any output but divided for transport to meet the most stringent transport limits imaginable.

The authors also state that, as a result of such machines being taken to site in pieces and then erected at the site for the first time, "certain tests which it is practicable to carry out only in the manufacturers' works cannot be made." I do not agree. Many machines to be erected on site in the way which they describe have, in fact, been erected and tested in the manufacturers' works. That has been particularly the practice in this country. In this class of business, of course, until quite recently all our machines went abroad, and many owners consider that the test of machines in the works has a value in that it gives the operating staff confidence in the machines. It has been quite usual to test one machine out of a number, and the cost of doing that is relatively very slight.

In the same Section of the paper the authors draw attention to the fact that in the speed range of 400–600 r.p.m. the attainable generator output is well above what is at present required. A challenge comes to the generator manufacturer in the next higher speed range, 750–1,000 r.p.m. I should like to comment on a dangerous practice that is met with here and there, of attempting to raise the upper limit of attainable output of a 750–1,000 r.p.m. generator by allowing the measured first critical speed to be below the runaway speed. There is no true parallel between a water-turbine-driven machine and a steam-turbine-driven machine, because when a water-wheel runs away it is power drive and may linger at the first critical speed where there may be resonance leading to serious damage. A much better way of dealing with the problem is one which is being pursued by turbine makers, and consists in artificially reducing the maximum speed for which machines have to be designed.

It is unfortunate that the authors have chosen as a basis for Fig. 7 the phenomenal rise of prices during the last half century. This was greater than in the previous 300 years and I think a much smaller rate of increase would have been more realistic.

In Table 3, scheme (*a*) includes about 45% greater reservoir working capacity than scheme (*b*), yet the capital cost is affected to the extent of only about 2%. If these figures are correct it would seem that the cost of reservoir working capacity has little effect on the total cost. Does this not have an important effect on the conclusions to be drawn?

Mr C. W. Marshall: My long experience of hydro-electric developments in Scotland leads me to draw attention to the fact that early economic studies usually resulted in deferment of construction with consequent increases in capital cost and loss of irreplaceable fuels. A noteworthy exception to this rule is the Galloway scheme, which was constructed at an opportune time and has been a conspicuous success.

Regarding the present electricity supply situation in South Scotland, it is useful to note that about 30% of the total power requirements were derived from hydro-electric sources, about two-thirds of which came from the north of Scotland system. It was frequently possible to provide all the night load requirements by hydro-electricity alone, although for security we operated a few of the most economical steam sets to safeguard the industrial areas.

A minor point of practical interest to operating engineers is that when the highly efficient modern steam sets in Glasgow were shut down for several hours during the night, the noise emission was of the order of 100 phons. This caused serious disturbance to the would-be sleepers for a wide area round the station. The noise trouble was, however, completely eliminated by application of an extremely efficient spray silencer devised by the superintendent of Braehead power station. It is also important to realize that the sporadic starting and stopping of steam plant is expensive in fuel and plant maintenance.

Hydro-electric plant is almost completely free from these disadvantages and I should like to record a tribute to the designers and constructors of the Galloway scheme, which has now traversed almost every weather condition that has been encountered in the area. I especially wish to associate that tribute with the names of William McLellan and James Williamson.

There is only one respect in which a slight improvement could be effected, so far as my experience shows: the rates of rainfall and of run-off from the hillsides into the reservoirs can be phenomenally high, with the result that the siphon reliefs may come into action; and as these cannot be stopped until the water level has fallen sufficiently, flooding may result. It should be possible for the authors, who are familiar with balanced systems of protection, to devise a hydraulic analogue of the Merz–Price system which would set this to rights.

On the general question of hydro-electric machine design, I find it difficult to accept the authors' contention that the theoretical basis is as unsatisfactory as they have indicated. I feel sure that the water-turbine designers and manufacturers have a firm basis from which to work, otherwise they could not have achieved the remarkable success which they have obtained.

The subject of testing hydro-electric machines has always been controversial, and I have so far been unable to obtain a code of testing practice for which our research advisers will vouch. I attach great importance to this matter, for although I agree with Mr Lawrie as to the showroom value of Scottish hydro stations, I suggest that they could be of much greater value as hydro-electric research centres. The Galloway stations are particularly convenient in this respect, and measurements made on the plant there could be more convincing to potential customers than any laboratory-scale tests.

Mr H. Headland: Oversea, remote water-power projects and lack of fuel resources often go together, whereas in Britain the cost of thermal generation is the criterion. The theme is therefore the economic hydro-electric/thermal plant ratio for different conditions.

Although the authors define firm annual load factor in terms of the driest year, requirements may be modified by analysis of drought frequency and duration or on a probability basis, since supply reliability in under-developed areas need not initially approach that demanded in industrial countries.

The points concerning optimum installed capacity might be illustrated by developments in Scotland, where initial schemes were designed for low load-factors with subsequent stations for higher load-factors, in contrast with which we find Owen Falls and Kariba being developed for load factors of 60–70% and joint thermal and hydro-electric schemes in the Middle East because of river flow characteristics.

In Britain, power demand follows seasonal run-off, but in Switzerland, where run-off depends on snow melt, there is a standardized procedure gives seasonal values for energy. It might be worth considering whether this concept is applicable in Britain.

There are few statistics concerning hydro-electric plant availability, but on one large system 98–100% is common and values below 95% are unusual. An American I.E.E. committee has given a figure of 0·46% of the exposed time for forced outages.

The suitability of hydro-electric plant for fluctuating loads may depend on the surge chamber being capable of absorbing severe load increases. System characteristics may change during the life of a station, and a tendency to reduce surge-chamber dimensions requires investigations to define design conditions more closely than seems possible at present.

Dr J. H. Walker: The paper emphasizes the increased capital cost of hydro-electric stations as compared to steam stations. In this connection some reduction in the cost of machines for the former could be obtained by eliminating acceptance tests in the factory. The practice of carrying out acceptance tests on site is widespread in the United States and Canada; in addition to being covered by an appropriate test code, it is normally included in specifications issued by American operating companies and consulting engineers. This not only reduces costs but also shortens delivery times and releases valuable factory space. The disadvantage, of course, is that testing in the factory can be carried out with greater accuracy, and in addition the designer is denied the opportunity for experimental tests required for development work.

On the question of turbines, the authors' opinion would be interesting on the use of the propeller type, which is very similar to the Kaplan but much cheaper, with fixed blades and substantially the same full-load efficiency. It is applied to a large extent in America in hydro-electric stations having a total output of several hundred megawatts, so that the efficiency of the station is maintained by keeping all running sets on full load and shutting the sets down as the load on the station falls.

It would be of considerable interest to engineers in this country if the authors could give their views on the extent to which the power from the Severn Barrage, when built, could be co-ordinated with the cross-Channel power link with France.

Mr D. J. Bolton: In Section 11, referring to the pumped-storage scheme, the authors state that every kilowatt-hour generated represents a definite financial loss. Perhaps we can apply that phrase to any storage scheme by saying that every kilowatt-hour provided for by way of storage represents a definite financial loss; in other words that the cost of impounding—so far as it is a kilowatt-hour cost—is broadly more than the fuel and other running costs of a steam station. On the other hand, as the authors point out in Section 2.1, the kilowatt cost of a hydro-electric scheme, or the incremental cost of increasing the plant capacity, is generally less than with a thermal station. Putting those two together, we get the equation that the lower the design load-factor (other things being equal), the more economic is the scheme likely to be, so that it is just a question of what is the lowest load-factor which can be accepted; in other words, what is the minimum number of kilowatt-hours to be associated with a kilowatt in order to give it the proud title of a firm kilowatt?

In estimating this I am not sure whether the authors have given sufficient credit to hydro-electric plant for its operational elasticity. A curve similar to that in Fig. 2 was given by Cooper as Fig. 10 in his paper on load dispatching.[20] It shows a load/duration curve, and side-by-side are rectangles showing the difference between the ideal and the actual. The rectangles at the bottom show the deficiency of kilowatt-hours as compared with the ideal, and the rectangles at the top the excess of kilowatt-hours which had to be generated by low-merit plant. Any station or system engineer, with that curve in mind, if he were offered a peak-load steam station with only enough coal for 200 hours a year, would say that it was little use; but if he had a hydro-electric station with only enough water for 200–300 hours a year, would he necessarily make the same reply?

Mr Cooper attributed the divergence at the top of the curve almost entirely to the lack of operational flexibility, such as unsuitability for 2-shift working, quick starting and stopping, and to extensive running of low-merit plant to provide security of supply. I suggest that the position may be different with hydro-electric plant. In Mr Cooper's and the present authors' curves the top 10% of the load lasted for 1½–2% of the time. If we apply that to our present peak of 16,000 MW and hold to the ideal curve, we shall have about 1,600 MW of plant with a load-factor of about

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[20] A. R. Cooper, "Local Dispatching and the Reasons for it, with special reference to the British Grid System." J. Instn Elec. Engrs, vol. 95 (1948), Part II, p. 713.

$1\frac{1}{2}\%$, i.e., about three times as much as the whole of the hydro-electric plant in this country, including the north of Scotland. I do not suggest that we could keep to the ideal curve, but it suggests that, if we have a load in the country of the order of 1,600 MW persisting for only 150 hours a year, it provides us, with our present interconnected schemes, with the elbow-room to fit in some very low load-factor hydro-electric or pumped storage schemes.

The difference between the ideal and the actual operating times is worked out in Figs 2–5, and I have two criticisms to make of the method. The first is that it seems to be based entirely on typical steam conditions. The difference between the straight lines A–A, etc., in Fig. 4 and the curved lines arises because on many days it was necessary to have sets steaming and not necessarily running, in order to provide security. A hydro-electric station might have served the same purpose on many of these days without using a gallon of water.

The other objection to the method is its purely empirical basis. The authors suggest that the same percentage of any peak should always be supplied by the hydro-electric plant. If the peak of the year is taken as 100 units and the hydro-electric plant supplies 1%, it supplies 1 unit, and if on some other day the peak is 70 the authors suggest that the hydro-electric plant should provide 0·7 unit. I wonder why? There is then a margin of steam plant; why call on the hydro-electric plant? The fact that the latter is there ready to come on means that it is in quite a different position from the steam plant with which it has been compared.

Mr K. H. Tuson: I endorse the requests for more information on Fig. 6, and suggest that the authors might give the form of the annual load curve on which this figure is based. I should also like them to set out the model formulae which are represented by the dots A–G in Fig. 8.

Turning to Section 4, I think that the first characteristic which the authors give of nuclear energy, namely that power plants must be at a very long distance from centres of civilization, is a purely passing disadvantage and that the dangers are much exaggerated in the public mind. A reactor at Chalk River broke down or exploded without any very disastrous results, although the outage was of extremely long duration. I do not think that it is fair in computations to load nuclear energy with the cost of unduly long transmission lines.

The figure of 1d. per kilowatt-hour for the cost of nuclear energy has been mentioned several times and is referred to in the paper. Have the authors any information on the make-up of this figure? My belief is that it does not give sufficient weight to the cost of the fuel (because it comes from Government establishments), to the cost of processing and ash disposal and, in the case of breeder reactors, to the cost of extracting the plutonium. Until some authority publishes a reasonably detailed estimate, little weight can be given to the *ex cathedra* pronouncement of overall figures.

Mr V. G. Newman: The principles set out by the authors for the integration of hydro-electric with thermal generating plant are broadly in line with those followed by the B.E.A. The characteristics of suitable hydro-electric sites in the Authority's area are such that development can most profitably be carried out for peak-load generation at annual load-factors of the order of 10–20%, and an economic comparison with alternative coal-fired plant must take into account the much higher load-factor at which the latter would operate during most of its life.

The improvement in average system efficiency which results from the addition of new thermal plant, which is sacrificed when capital is allocated instead to hydro-electric construction, is greatest during the first year of operation, when the new station is one of the most efficient on the system and operating at its maximum load-factor. Consequently, if it can be shown that the hydro-electric scheme will be financially attractive during its first year of operation, its economic soundness throughout its life will be established.

A careful study of the full effects of a pumped-storage scheme on the operation of the system will often show that the fuel saved as a result of the displacement of thermal plant, which would otherwise have to be reserved for peak-load and standby duties, more than offsets the additional fuel consumed in supplying the pumping energy. Moreover, with gradual improvement in system efficiency, the pumped-storage scheme will require less coal year by year.

The authors do not appear to have taken into account in their estimates for a pumped-storage scheme items (*b*), (*c*) and (*d*) referred to in Section 2.3.2. From calculations recently made by the B.E.A. it would appear that, for the size of installation considered in the paper, these three items would entail a net reduction of some £50,000 in the estimated annual surplus.

Mr Lane has already mentioned briefly the pumped-storage scheme which the B.E.A. is at present seeking parliamentary powers to construct, and a few additional figures may be of interest. The operating head will be 1,000 ft and the total installed capacity 300 MW. Operation will be on a 5-day pumping cycle with generation over 4 hours a day. The daily load-factor will be about 16·7% and the annual load-factor about 11·4%.

A further project, a hydro-electric scheme, is also proposed. This is the Rheidol Scheme in Cardiganshire, which will develop an output of 49 MW in two main stages, having an aggregate head of 900 ft. The average annual load-factor is estimated at 19% and the storage capacity will correspond to some 21% of the average annual output. This will ensure that the installed capacity will be wholly firm, even in a dry year, although the energy output in such a year will fall to about 75% of the average. This is in very close agreement with Fig. 6 of the paper.

Dr T. P. O'Sullivan: I should like to refer to two points in the paper which are largely complementary. The first is in Section 5, where the estimated life of a hydro-electric station is given as approximately 60 years, and the second is in Section 2.1, where the available energy is stated to be fixed by run-off, as it very largely is. In that connection, however, I should like the authors' opinion on the possibilities of certain adverse factors affecting the available energy.

The first—perhaps the least important, although it applies quite forcibly in tropical countries—is evaporation, and the second is silting. In one instance with which I am familiar there is a relatively small reservoir involved with a dam of low head, and the evaporation is some 20% of the total capacity of the reservoir; this is in Northern Nigeria, where the dry season lasts for at least six months of the year. The maximum temperature there is approximately 105°F, but I understand that in certain other parts of the world, including areas of the United States, the Sudan and the Middle East, evaporation takes place at an even higher rate.

So far as I can gather, silting is not a great problem in this country, neither does it seem to have caused undue anxiety in Europe, South Africa and India; however, there have been some particularly troublesome cases in the United States. The Zuni reservoir in New Mexico, of 14,800 acre-ft, lost 20% of its capacity five years after completion, 25% in ten years and 63% in 15 years. At the end of 15 years it was decided that special measures should be taken in the form of silt traps and revetments to slow down the rate of loss, after which it was necessary to go further and put in an extensive sluicing system to set up erosion in the bed. It is claimed that, during a drought in 1941, a group of power operators in the southern states of America lost 90,000 MWh through silting.

I should like the authors' comments on these factors, which may become of some considerable importance in connection with works in certain oversea locations.

Mr S. C. Brealey (*communicated*): I have just been engaged in examining a project which includes thermal generation at an open-cast lignite mine site overseas and integration of the station into a predominantly hydro-electric system. In this case it is obvious that the economic comparison must be drawn between a hydro-electric station on the one hand and a combined mine and power station on the other. The generation cost in each case consists solely of labour, stores, depreciation and other capital charges. In the thermal scheme, fuel is an internal transaction only. The capital cost of the combined mine and power station is comparable with that of new hydro-electric schemes. In this case, therefore, on the same basis as Fig. 7 and allocating 60 years' life to the initial development cost of the mine, the additional benefit which the owner could obtain by installing a hydro-electric station is very considerably less than the authors indicate. Furthermore, no additional benefit at all would accrue until 30 years had passed, and it is doubtful whether it would then be sufficient to influence materially a choice made at the present time.

The additional benefit to the owner according to the authors is the shaded area in Fig. 12a while according to the above it is as shown in Fig. 12b.

It may be argued that this would not affect a purely electrical undertaking, which would not usually own its own mines but, of course, the depreciation and other capital charges on the mine are included in the price of fuel. Where fuel is obtained from a number of mines and where the price structure is unified, these effects are masked, but they are nevertheless still present. During the first 30 years such a thermal power station would benefit by the lower capital charges on mines constructed before the commissioning date (as would all other fuel users), and during the second 30 years the reverse would be true (assuming an average of 60 years' life for all the mines involved in the fuel price structure).

The point is that, when comparing the economics of hydro-electric and thermal generation (as distinct from stations), the economics of the mine should not be left out, and this is particularly true in countries where public control of the mines and power stations demands consideration of the wider economic ramifications. In this country, under present conditions of coal scarcity and restrictions on capital expenditure, the mine must be considered with the power station when it is remembered that a 1,000 MW station at 50% load factor consumes coal at the rate of about 5,000 tons per day; this is the output of a large colliery, and therefore a colliery must be sunk somewhere to supply that station.

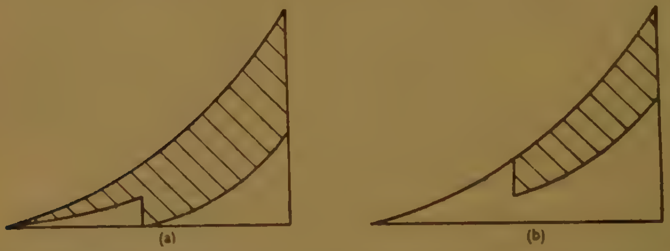

FIG. 12.—BENEFIT TO OWNER DUE TO RISING PRICES. SHADED AREAS SHOW ADDITIONAL BENEFIT DUE TO HYDRO-ELECTRIC GENERATION

(a) Mine neglected. (b) Mine included.

A further point which operates against the case for hydro-electric stations is the fact that an installed kilowatt capacity of hydro-electric plant is very often not as valuable as an installed kilowatt capacity of thermal plant, because the latter is more firm.

A number of speakers emphasize the saving of coal which hydro-electric stations bring about, for instance in Scotland, but in some cases it can be the other way around with thermal stations bringing about a saving in water. This is often the case where generation is secondary to the demands of irrigation and where rainfall is irregular. In these cases the ratio of firm to total capacity is very low and may even be zero, and the value of an installed kilowatt capacity is correspondingly reduced.

One must therefore conclude, from these considerations and from the authors' admission that many factors have been omitted, that the conclusions in Section 5 have little value. This is not a plea for more thermal stations, but a plea for treating each system on its own merits without over-simplified generalizations. If one can generalize at all it can be said that hydro-electric generation has the advantage over thermal generation in reduced labour and stores costs at the expense of reduced firmness.

Mr E. M. Johnson: In the very early days of electricity supply the view was sometimes held that there was no future for electricity until means could be found of storing it. Fortunately, the handicap has not proved as serious as was thought. The reason for this has been the comparative abundance of the prime sources of energy. Nevertheless, there is some truth in the old view ; rising costs and increasing scarcity of sources of energy will give added point to it.

The authors deal with their subject in general terms, and I think it is unfortunate that the title of the paper and a part of the presentation of it give an impression of a much more restricted review of the subject than that which the authors in fact present. The principles of the method of approach which they elaborate are applicable to any mixed power system, whether having pumped storage or not. The latter is merely a special and very important example of a possible component of a mixed system.

That the operation of mixed systems provides advantages not available from one alone is, of course, not new. So far as I am aware, however, the paper is the first published analysis of the basis governing the development of such systems. A very early example was the interlinking of power systems having low-head plants with others operating at high heads which took place in Switzerland immediately after the 1914–18 war. The addition of pumped storage was being discussed even in those days.

The authors, of course, go much further and include even the "installation of thermal plant especially for firming the water power." I think, therefore, that they may well claim to have dealt, not only with the analysis of all the factors governing the economics of mixed power systems (of whatever sort), but also to have issued a kind of challenge to power-system operators to endow their systems with precisely the characteristics they require by mixing the types of generating plant and, if advantageous, by adding pumped water storage even at low efficiency.

Mr H. Headland : Section 2.3.2 is an admirable summary of the effect of hydro-electric plant on overall fuel consumption. This problem also occurs in a different form for tidal-power plants with or without pumped storage. The difficulty is to forecast future trends of the factors concerned, and similarly the question of increasing plant capacity requires determination of the expenditure which can be justified to cope with future conditions, especially where tunnels, surge chambers and pipe-lines are involved. These points emphasize the element of judgement entering into hydro-electric station design, but nevertheless it should be feasible to devise approximate economic criteria to take the appropriate factors into account. The authors might also have mentioned that it is sometimes economical to increase the plant load-factor by diversion of adjacent catchments to meet the needs of an expanding system.

It is perhaps fair to mention that several studies of firming with pumping plant suggest that money can be better invested in increased storage, even though the economic height of the dam might appear to have been reached. Short-term weather prediction is now an established facility in hydro-electric system operation, and much research is being devoted to the long-term aspect. With the former it would seem that better use might be made of firming by pumping by allowing greater latitude in operation of storage capacity, particularly if there is some diversity of run-off in several catchment areas.

The authors are to be commended for an authoritative statement on the effect of nuclear power in relation to hydro-electric development. This should dispel much loose thinking on this question by those unfamiliar with both sources of power.

The novel, but admittedly oversimplified, treatment of the rising-price problem reinforces the need for a long-term view of marginal projects such as the Severn Barrage, and a more realistic economic criterion than that used in the 1945 Report when coupled with rising coal costs and other factors which have changed in the meantime.

It is unfortunate that no reference is made to outdoor, semi-outdoor and underground stations or to new arrangements of turbines and alternators, including machines where the conventional spiral casing and draft tube have been simplified, with the alternator housed in a submerged chamber. Such developments have resulted from the need to economize in space and expenditure, and are perhaps more important than the details to which some attention has been devoted.

The statement in the penultimate paragraph of Section 8 should not be accepted by plant designers with complacency. In low-head and tidal-power schemes, civil-engineering and plant costs may be about equal. The fixed annual charges on the former are about 3·74% against 4·86% on the latter, and reduced plant costs, perhaps bringing cheaper civil-engineering works in their train, are worth seeking on economic grounds.

The paper encourages the application of reversible Francis pump turbines, but passes over the operating head without comment. It is claimed in the United States that these can be designed for heads exceeding 1,000 ft, and while this is hopeful, no experience is available to provide convincing evidence that a single turbine runner can be used where experienced pump designers would undoubtedly employ 2-stage impellers.

The reference to the quality of steel castings should have the unqualified attention of steel founders in this country, where it is apparent that the casting technique for runner and other components for hydraulic machines has not received the attention which it deserves. On the whole, welded construction has proved satisfactory, but among hydraulic engineers and steel founders there is a need for acceptance of radiographic and other non-destructive methods of testing as a routine rather than as a procedure to be adopted when trouble is discovered.

While there are undoubted merits in testing model turbines, they will not have universal acceptance until prototype/model similitude can be assured and scale effects for partial loads are resolved. Current empirical or semi-empirical formulae predict peak efficiency, but a recent approach by Hutton[21] covering full and partial load performances of Kaplan turbines merits the attention of those who have model and prototype test data to check it.

The inter-territorial character of interconnected transmission might also have been associated with difficult problems arising in Europe, Africa, America and Canada, where water-power development on rivers affecting sovereign states have had, and will have, to be solved.

Mr E. W. Connon: Twenty-five years ago it was generally considered that hydro-electric stations, because of their high capital cost, had to work at a high load-factor, and the paper demonstrates how, as a result of the appreciation that the capital cost is generally related more to the energy than to the power, the trend has been reversed. To-day, the quick-starting characteristics of hydro-electric plant, enabling the stations to be used to assist at times of rapid load rise and of unexpected plant shortages, are those which are perhaps most valuable in the operation of interconnected systems. The pumped-storage scheme is a logical outcome of these requirements.

When discussing the relative merits of 2- and 3-machine installations for pumped storage, the authors do not appear to mention the problem of starting, which would seem to increase considerably the electrical costs of the 2-machine installation, since hydraulic starting cannot be used. Can they say what methods are proposed, and what they will cost?

(*Communicated*): It seems to me that a simpler way of considering the rising-price phenomenon is to say that the rate of interest is, in reality, reduced by the price rise if the real value, in goods and services, is considered. Anyone who has compared the purchasing power to-day of a Savings Certificate bought, say, in 1939 with that of the cash with which it was bought will appreciate this.

Mr J. F. Dunn: With regard to the welding of pressure parts, it is in my experience noteworthy that many Continental manufacturers adopt a

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[21] S. P. Hutton, "Component Losses in Kaplan Turbines and the Prediction of Efficiency from Model Tests." Proc. Instn Mech. Engrs, vol. 168 (1954), p. 743.

100% radiographic control technique—which means that every inch of important welding runs is photographed. These records are carefully examined by persons qualified in this respect, and any defects which are revealed and which are judged to be below a certain very high acceptance standard are cut out and repaired. The records are then stored for possible future reference. This very thorough non-destructive testing procedure is emphatically stated by every manufacturer who practises it to pay high dividends, and there is no doubt also that it has a profound psychological effect upon visiting inspecting engineers.

Messrs T. G. N. Haldane and **P. L. Blackstone** (*in reply*): We are glad to have Mr Lawrie's support on a number of matters covered in the paper. All who have been concerned with hydro-electric work will agree that the progress which has been, and is being, made in the North of Scotland is of immense benefit in helping to promote British hydro-electric work overseas.

In answer to Mr Mountain, we consider that under normal circumstances the amount of storage required, and hence its cost, is affected only slightly by the design load factor. We agree that hydro-electric plant is certainly more suitable for standby purposes than thermal plant, bearing in mind the coal consumption which must continue when thermal plant is kept available but not generating. This is particularly true of pumped-storage plant, and it may well be that, if such plant is used largely for standby purposes, the annual consumption of pumping energy will be less than usually shown in calculations, such as in Table 3.

The expression "firm power" does require some amplification, as Messrs Hunter and Fulton suggest, and a working definition might be that hydro-electric power can be regarded as firm if it is capable of development in the worst conditions which can arise in 30 years; but, of course, still worse conditions may occur over some longer period. There must always be some margin to deal with the completely abnormal year, or alternatively, readiness to accept some restriction under very rare circumstances. No system can ever have 100% reliability, and in some countries such may not be necessary. How much additional expenditure is justified in approaching closer to 100% reliability must be a matter of judgement. An all-thermal system may not be completely reliable, owing to possible restrictions in the supply of fuel due to weather and other causes. We therefore see no reason to agree with Mr Brealey that a hydro-electric kilowatt is necessarily less valuable than a thermal kilowatt on grounds of reliability.

Mr Lane says that some B.E.A. stations operate at annual load-factors of 5% or less. We wonder whether the explanation may not be that such very low load-factors occur when thermal plant is used largely for standby purposes. It may be that there are not many thermal stations in regular, as opposed to standby, use which have load factors much less than 15%.

Mr Lane also refers to the load factor of future nuclear plant. To avoid running such plant at reduced load-factors, which would be uneconomic and technically difficult, we visualize the possibility of nuclear plant operating in conjunction with pumped-storage plants, so as to maintain the load factor on the former as high as possible.

As both Mr Lawrie and Mr Fulton point out, there is a concealed benefit accruing from expenditure on hydro-electric plant because of the saving of coal. We feel this can best be regarded as being due to the difference between the home prices of coal and the higher price of imported coal. It must, however, be borne in mind that some saving of coal may result from expenditure of the same capital on new thermal plant instead of the alternative hydro-electric plant.

Mr Fulton has made the interesting point that the justifiable installed capacity of a hydro-electric station can be related to the rate at which the all-in kilowatt-hour price of the B.E.A. Grid tariff increases with decreasing load-factor. For practical purposes in this country, such a criterion would be very suitable provided it can be assumed that the Grid tariff accurately reflects the true costs of generation. But, of course, it still remains necessary to show that the low-load-factor hydro-electric plant can be fitted satisfactorily into the adjusted load/duration curve.

Fig. 6 has been commented on by Messrs Fulton, Hunter and Tuson. It is one of many such curves which can be drawn from a study of mass run-off curves for different catchments. Its shape will vary according to the ratio of maximum to mean monthly demand assumed. The curve shown is an average for several different catchments in Scotland, and was intended rather to illustrate the principle of firming of power by pumping than to be taken quantitatively. We wonder whether Mr Fulton's curve (iii) in Fig. 11 perhaps applies to one particular catchment.

In reply to Mr Tuson, the estimated cost of nuclear power of 1d./kWh was taken from statements published before the issue of the Government's White Paper, which (taking account of the probable value of the plutonium by-product) assumes 0·6d./kWh. While experience of the accident at Chalk River was reassuring, it is probable that reactors less inherently safe than those at present being built will eventually become the most economic, and that it will be considered necessary to site such reactors in fairly remote areas.

We agree with Dr Walker that propeller turbines still have their uses, provided that the relatively high drop in efficiency at loads above and below the optimum can be justified by saving in initial cost. This will more easily be done in large multi-unit stations running on base load.

In answer to questions raised by Messrs Fulton and Marshall, the criterion for model-turbine or site-efficiency tests is: Which will give the more accurate and comprehensive results consistent with reasonable expenditure, time and inconvenience during testing? It seems that, if the answer to this is in favour of homologous model tests, there is no justification for prejudice on the grounds that the actual prototype machines are not tested for efficiency. Certain tests can be made on models which are impracticable on site.

A new British Standard for testing water turbines is now being prepared. It is hoped to carry out some research into different methods of flow measurement at one of the North of Scotland Hydro-Electric Board's stations.

While we cannot wholly follow Mr Bolton's line of argument in which fixed storage costs are compared with steam running costs, we largely agree to his conclusion. In regard to the empirical method illustrated by Fig. 4, it must be borne in mind that at other than peak times there may not be surplus steam plant available because of outages for maintenance.

We agree with Mr Newman that the factors enumerated in Section 2.3.2 have to be considered in deciding whether or not to proceed with a pumped-storage scheme. As pointed out in Section 3, however, it is likely to be found that over a long period of time the total of the effects is zero.

We also agree with Dr O'Sullivan as to the importance of both evaporation and silting in certain circumstances. In extreme conditions silting might affect the assumed life of the works for sinking-fund purposes.

Mr Johnson suggests, in connection with Fig. 7, that the rise in prices during the last half-century is phenomenal. This must be a matter of opinion, of course, but there is no doubt that the fall in the value of money is still continuing; and whatever may be the rate during the next half-century, the effect illustrated by Fig. 7 is likely to be important.

The pumped-storage scheme, particulars of which are detailed in Table 3, is based on an actual project recently investigated. We agree with Mr Johnson that in this particular case the extra cost of increasing storage capacity was, owing to the nature of the ground, unusually low.

Mr Brealey raises a complex problem by taking into account capital expenditure on coal mines. We do not think this is relevant to Fig. 7, which relates simply to the normal capital expenditure incurred by the owners of hydro-electric and steam stations respectively. We have been careful to point out that financial gain accruing to the owners of power stations due to fall in the value of money is not necessarily gain to the nation. Nevertheless, the effects dealt with in Section 5 are of considerable importance, particularly to those financially responsible for expenditure on hydro-electric works.

With Mr Headland's remarks we are much in agreement, especially in his emphasis on the element of judgement entering into hydro-electric station design.

We are not in a position to deal here with the problem of starting of pumping plant raised by Mr Connon, but we agree with his comment on the purchasing power of savings certificates, and with Mr Dunn's remarks on radiographic control of welding.

We regret that lack of space does not permit us to reply to all of the many interesting points which have been raised.

Paper No. 6070

SETTLEMENT ANALYSES OF SIX STRUCTURES IN CHICAGO AND LONDON

by
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(*Ordered by the Council to be published with written discussion*)

SYNOPSIS

The Paper describes a development of the existing methods of calculating settlements of structures founded on clay. It is applied to the analysis of the settlements of three buildings in Chicago, founded on the soft normally consolidated clay of that city; also to the analysis of two bridges and a building on the stiff over-consolidated London Clay. The three Chicago buildings are large structures constructed about 1890 and the settlement records cover a period of 20 to 60 years. The two London bridges which span the Thames at Chelsea and near the City, have detailed records extending for about 15 years; whilst the building, situated at Elstree, has small concrete footings and the settlements were observed for 4 years. In all cases the settlements have reached almost their final value and thus, with the wide range of foundation size and the two very different clays, the records provide a good basis for comparison with theoretical calculations; especially since there is a considerable amount of data on the geotechnical properties of the clays in Chicago and London. The comparisons show that the settlements, both final and at the end of construction, can be computed with errors not exceeding the limits of about −30% and +50%.

SETTLEMENT ANALYSIS

Definitions and aims of settlement analysis

ON opening an excavation for a foundation the pressure acting on the soil below the excavation is reduced and hence the soil will heave upwards. This heave is shown as ρ_h in Fig. 1. When the structural load becomes equal to the weight of soil removed the settlement is roughly equal to ρ_h, the original stress state having been restored. At this stage the foundation is fully buoyant and the net pressure is zero. Further structural loads increase the stresses in the soil beyond their original values and it is the corresponding "net settlements" resulting from the increase in net foundation pressure that are calculated in the methods of settlement analysis. At the end of construction the net settlement is made up of two parts: the "immediate" settlement due to deformations of soil taking place without change in water content (and hence without volume change in a saturated clay); and the "consolidation" settlement arising from extrusion of some of the pore water from the clay (and hence a volume decrease). Owing to the low permeability of clays the amount of consolidation at the end of the construction of a building is usually small and, as an approximation, it may often be neglected.

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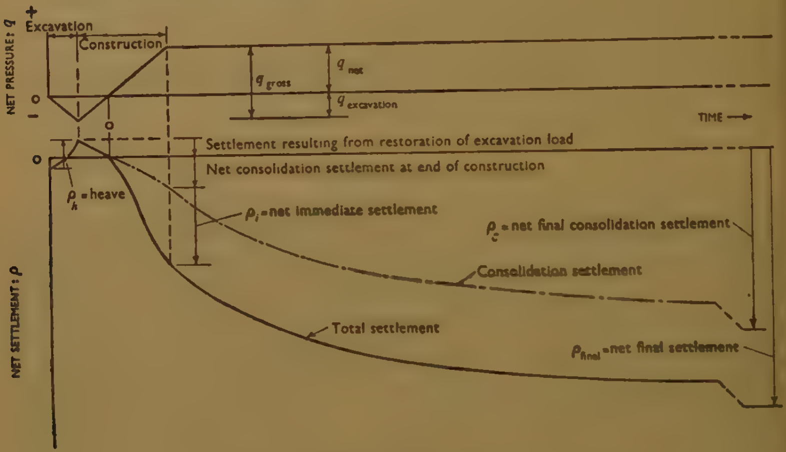
NET PRESSURE: q
+
Excavation
Construction
0
−
q_{gross}
q_{net}
$q_{excavation}$
TIME →
0
0
Settlement resulting from restoration of excavation load
Net consolidation settlement at end of construction
ρ_h = heave
ρ_i = net immediate settlement
ρ_c = net final consolidation settlement
Consolidation settlement
Total settlement
ρ_{final} = net final settlement
NET SETTLEMENT: ρ

FIG. 1.—FOUNDATION SETTLEMENTS
GENERAL DEFINITIONS

In the course of time, however, the consolidation settlement increases until all the excess pore-water pressures set up by the net building load are dissipated. This process generally requires for its completion a period of years or even decades; the net "final" settlement is typically between two and six times the settlement at the end of construction. Strictly speaking it is not possible to refer to the final settlement of a clay as an absolute quantity, since the time/settlement curves usually approach a slightly inclined asymptote, and not a horizontal asymptote as would be expected from consolidation theory. This slow continued settlement is referred to as "secondary compression" and in some clays may be of importance. In the present cases, however, it seems to be a minor component and certainly of little structural significance.

The aims of a settlement analysis are therefore to estimate:

(1) the settlements during replacement of the overburden pressure, i.e., the settlements under zero net pressure. The data on this matter are still insufficient to permit any general

conclusion to be drawn and no further reference to the problem will be made in this Paper ;

(2) the net final settlement ;
(3) the net immediate settlement at the end of construction ; and
(4) the rate of settlement during and subsequent to construction.

Calculation of net final settlement

The usual method of computing net final settlement is based on the simplified assumption that this settlement is equal to the consolidation of a vertical column of clay beneath the foundation. The first step is to calculate the effective vertical stress σ_0 in the ground before building operations commence. The increase in vertical stress $\Delta\sigma$ at any depth z below foundation level, due to the net structural load, is then calculated from elastic theory,[1–5] and the net final settlement is :

$$\rho_{\text{final}} = \int_0^Z m_v \,.\, \Delta\sigma \,.\, dz \qquad . \quad . \quad . \quad . \quad . \quad . \qquad (1)$$

where m_v is the oedometer compressibility of the clay at this depth as measured in the standard laterally-confined consolidation test, and Z is the full thickness of the clay strata.

For normally-consolidated clays it is necessary to correct the pressure/void-ratio curve from which the compressibility is deduced, and the accepted manner of making this correction has been described by Terzaghi and Peck.[6] For heavily over-consolidated clays, where the pre-consolidation load is ten or more times the existing overburden pressure, the sensitivity is low and sampling disturbance probably has little effect on compressibility. For such clays the compressibility can be taken directly from the test curve. With lightly over-consolidated clays a correction is necessary, especially where the applied net pressure from the building is small, but the best form of the correction is still in doubt. The procedure suggested by Schmertman[7] points towards a solution of this problem.

The method of analysis expressed by equation (1) was originally proposed by Terzaghi [8, 9] for calculating the settlement of a layer of clay subject to appreciable lateral restraint as, for example, a clay lying between beds of sand. In this case, and particularly if the layer is thin compared with the width of the loaded area, equation (1) is a close approximation. This method has, however, been extended to cases where the clay layer is relatively thick and even where the foundation rests directly on a deep bed of clay. Under these conditions there are important lateral deformations (causing the so-called " immediate settlement ") in addition to consolidation. Sometimes equation (1) has been taken, in such cases, as giving only the consolidation settlement, and to this the immediate settlement has been added ; whilst by other engineers equation (1) has been assumed to give at least an approximate estimate of the net final settlement.

[1–5] The references are given on p. 543.

An examination of twenty case records,[10] where comparisons can be made between the actual final settlement and the calculated settlement, shows that the second of the two assumptions mentioned above is the more correct. It is perhaps surprising that, in fact, the maximum errors in these twenty cases, involved in taking equation (1) as giving the net final settlement, do not exceed the limits of -27% and $+57\%$; whilst the average error is only $+5\%$. Theoretical considerations of this problem are difficult and have not yet been completed. Nevertheless an analysis[11] does suggest that equation (1) over-estimates the consolidation settlement of a thick clay layer by an appreciable amount, and that (in accordance with the field evidence) the settlements given by this equation are of the same order of magnitude as the combined consolidation and immediate settlements.

Owing to these circumstances and to the very simple nature of the calculations involved, equation (1) therefore provides a ready means of estimating the net final settlement, although it cannot be expected, in general, to give a correct estimate of net consolidation settlement.

It may be noted that the stress $\Delta\sigma$ is usually computed on the assumption that the structure is flexible, but quite often the condition of rigidity is more nearly approached. In such conditions corrections should be made to the stress distributions or settlements. Also, where the depth of a foundation is appreciable the settlements will be less than those resulting from the application of the same net pressure on the surface. As an approximation, the reduction in settlement due to this "depth effect" can be estimated from the curves published by Fox,[12] although there are theoretical limitations to the validity of these consequent upon the assumption of continuity of the soil above foundation level.

Calculation of net immediate settlement

Where the clay beneath a foundation is reasonably homogeneous, the net immediate settlement can be calculated from the standard equation in the theory of elasticity[1, 13, 14]:

$$\rho_i = q_n \, . \, B \cdot \frac{1 - \nu^2}{E} \cdot I_\rho \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

where q_v denotes the net foundation pressure, B the width of the foundation, E the Young's modulus of the clay, and ν its Poisson's ratio. For the problem under consideration ν is equal to 0·5, since for saturated clays only those settlements occurring with zero volume change are in question. I_ρ is the influence value pertaining to settlement and its magnitude depends upon the ratios L/B and Z/B, where L denotes the length of the foundation and Z the thickness of the underlying clay. The value of I_ρ in any given case can be readily computed from data given by Steinbrenner.[15, 16] Here, again, if the foundation is rigid and if it is at an appreciable depth below the surface, corrections have to be applied, as previously mentioned.

Young's modulus is measured as the slope of the stress/strain curve in an undrained compression test. In practice the stress applied by a building is a small fraction of the ultimate, and the stress/strain curve of a good undisturbed sample is usually linear within this working range. The value of E is, however, reduced even by a quite small disturbance in sampling,[17] especially in normally-consolidated clays; in consequence it must be anticipated that the immediate settlement will, in general, tend to be over-estimated.

Calculation of rate of settlement

If ρ_c is the net final consolidation settlement (equal to the net final settlement minus the net immediate settlement ρ_i) then the consolidation settlement at any time t after application of the load is:

$$\rho_{ct} = U . \rho_c \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

where U denotes the degree of consolidation. By Terzaghi's theory U is a function of the time factor $T_v = c_v . t/H^2$; c_v being the coefficient of consolidation as determined in oedometer tests and H the drainage path. Curves of U and T_v have been given, for the case of one-dimensional consolidation, for a range of stress distributions and boundary conditions.[16, 18] Consolidation is, however, almost always three-dimensional in clays beneath structures and in some cases the errors of using simple one-dimensional theory cannot be ignored. At the present time curves of U and T_v have been given only for a restricted range of conditions of three-dimensional consolidation [19] and the application of this theory is therefore still rather limited.

In the majority of settlement analyses that have been published it has been implicitly assumed that the final settlement given by equation (1) is an estimate of the final consolidation settlement and the immediate settlement has been ignored. Thus, in such analyses, the net settlement at any time t is taken to be:

$$\rho_t = U/\rho_{\text{final}} = U(\rho_i + \rho_t) \quad . \quad . \quad . \quad . \quad (4)$$

Calculations made in accordance with equation (4) will be referred to as the "conventional method" of settlement analysis.

The correct expression for the net settlement at any time after construction is, however, given by the equation:

$$\rho_t = \rho_i + U . \rho_c \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

Hence, since U is always less than unity during consolidation, the settlement given by equation (4) is always less than that given by equation (5), although the two equations naturally give the same value for final settlement when $U = 1$, and this final settlement is that calculated from equation (1).

CASE RECORDS

Fire Testing Station, Elstree

The Fire Testing Station, which is a reinforced concrete structure 138 ft × 36 ft in plan, was built near Elstree, north of London, between April and August 1935, to the designs of Mr R. T. James. The superstructure is supported on mass-concrete footings 5 ft × 10 ft in area at a depth of about 7 ft in brown London Clay, the footings being spaced at 15-ft centres. This clay has been proved to a depth of 15 ft below the footings, and it is probable that shortly below this depth there is a change to the stiffer blue London Clay. The clay is heavily over-consolidated, with a sensitivity of 1·0. Tests on undisturbed samples show that the average compressibility of the clay is 0·017 ft^2/ton and that Young's modulus is 100 tons/sq. ft. The coefficient of consolidation has an average value of 5·0 ft^2/year. The gross pressure on the clay is 1·1 ton/sq. ft (including the weight of the footings); allowing for the weight of soil removed by excavation, the net pressure is 0·8 ton/sq. ft.

Settlement observations by the Building Research Station began when the footings were placed, and continued for 4 years. No measurements of heave were made, but in view of the shallow excavation and the very short time during which the pits were open the heave must have been negligible. The time/settlement curve, averaged for seven typical footings, is plotted in Fig. 2 and it can be seen that although consolidation was not complete when the observations ceased the net final settlement must almost certainly lie between 0·6 and 0·8 in. It is therefore sufficiently accurate to take 0·7 in. as the net final settlement. The average net settlement at the end of construction was 0·3 in.

In calculating the settlements it has been assumed that the clay remains uniform to a depth of 20 ft beneath the footing, below which depth no further settlement will occur. For the final settlements computations have been made using equation (1), and the result is 0·85 in. Allowing for Fox's depth correction this is reduced to 0·65 in. For the immediate settlement elastic theory (equation (2)) shows 0·36 in., and again with Fox's correction this is reduced to 0·26 in.

It will be seen that, provided the depth correction is applied, very reasonable agreement is obtained between the calculated and observed final settlements. With regard to the rate of settlement, consolidation theory is applied to the difference between the final and immediate settlements; namely, it is taken that the final consolidation settlement is 0·39 in. Theoretical curves relating the degree of consolidation and the time factor T_v have been published for linear pressure distributions, and for the present case it seems reasonable to approximate the pressure distribution by a triangle of depth H, such that:

$$\tfrac{1}{2} \cdot q_n \cdot H = \int_0^{z} \Delta\sigma \,.\, dz = \frac{\rho_{\text{final}}}{m_v}$$

and, moreover, H in the above equation is assumed to be the effective drainage path. In this manner H is found to be 8 ft and hence $H/B = 1{\cdot}6$. Fortunately three-dimensional consolidation theory has been evaluated[19] for this particular case, and the resulting time/settlement curve, using equation (5), is plotted in Fig. 2. The agreement with the observed time/settlement curve is acceptable for practical purposes, and it may be noted that the calculated settlement at the end of construction is 0·4 in. as compared with the observed value of 0·3 in.

As a matter of interest the time/settlement curve as given by the conventional use of equation (4) is also plotted. It under-estimates the settlement at any time, but the errors are not great.

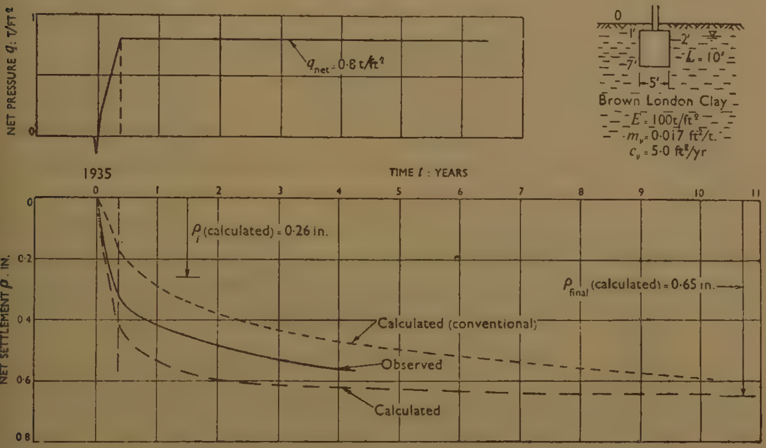

FIG. 2.—FIRE TESTING STATION, ELSTREE

Chelsea Bridge, London

The new Chelsea Bridge over the Thames is of the self-anchored suspension type with two towers supported on river piers spaced 350 ft apart. Details of the bridge, which was commenced in 1936 and completed early in 1937 to the design of Rendel, Palmer and Tritton, have been published[20] and settlements have been observed up to the present time. The piers are of mass concrete with base dimensions of 28 ft by 106 ft founded at a depth of 31 ft below the river-bed in the heavily over-consolidated blue London Clay. The gross pressure beneath them is 4·0 tons/sq. ft and the net pressure is 1·9 ton/sq. ft. For the settlement calculations this is considered to be reduced to 1·7 ton/sq. ft by skin friction along the side of the pier base.

From well boreholes in the neighbourhood it has been deduced that the

London Clay extends to a depth of about 60 ft below foundation level. Under the London Clay the Woolwich and Reading Beds, which consist of hard clays and sands, extend to a depth of a further 50 ft. These beds have been considered incompressible. No samples have been taken of the London Clay at the bridge, but experience indicates that this stratum is statistically very uniform over a wide area, and consequently it seems justifiable to use results obtained from tests on the clay at Waterloo Bridge[21] and also from a recent deep boring at Paddington.[22] At an average depth beneath the river-bed of 45 ft (equal to $D + B/2$) it is estimated that the compressibility of the clay is about 0·0075 ft²/ton and that the Young's modulus is about 350 tons/sq. ft. The coefficient of consolidation is estimated to be 3·5 ft²/year. The sensitivity of the London Clay is 1·0.

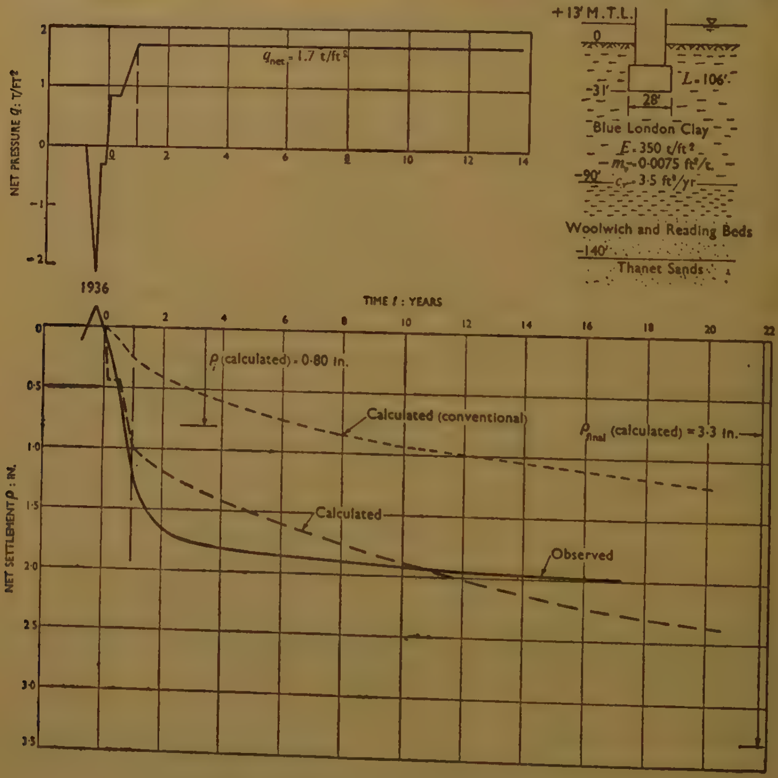

FIG. 3.—CHELSEA BRIDGE, LONDON

The observed settlements of the two piers are closely similar and the average settlement is plotted against time in Fig. 3. During excavation there was a heave of 0·25 in., and when the overburden pressure had been replaced the settlement was 0·15 in. The observations have been continued for 18 years and it seems that the movements have now practically ceased. The net final settlement can be taken as 2·1 in. and the net settlement at the end of construction was 1·2 in.

The settlement calculations have been based on the assumption that only the 60 ft of London Clay beneath the piers is involved, and the methods of calculation are exactly the same as those in the previous case. The computed net final settlement is 3·3 in., and the net immediate settlement is 0·80 in. (the depth correction being applied in both calculations). The calculated final consolidation settlement is therefore 2·5 in. and, using three-dimensional consolidation theory with $H = 43$ ft, the time/settlement curve obtained from equation (5) is that shown in Fig. 3. The calculated net settlement at the end of construction is 1·0 in., as compared with 1·2 in., and the calculated net final settlement is 3·3 in., as compared with 2·1 in. These results are in only moderate agreement and the actual rate of settlement in the first few years is considerably greater than the calculated rate. Nevertheless, the calculated settlements during a period of 20 years following construction are in error by not more than $\pm 0{\cdot}4$ in., or $\pm 20\%$ of the observed final settlement.

Again, as a matter of interest, the conventional method of calculation using equation (4) leads to settlements which are considerably too small—at least during this period of 20 years after construction.

Waterloo Bridge, London

Waterloo Bridge, the modern reinforced concrete bridge over the Thames which replaced Rennie's famous structure, was built between 1938 and 1941 to the designs of Rendel, Palmer and Tritton.[23] Borings and laboratory tests were made[21] prior to construction, and settlement records have been kept from the earliest stage of the work until the present time. The foundations of the piers resemble those of Chelsea Bridge. Settlement calculations, made in the same manner as described above for that bridge, have been published by Cooling and Gibson,[24] to whom the Authors are indebted for the results summarized in Table 1 and Fig. 4. In the former the average observed settlements of the four piers are given, whereas in Fig. 4 the time/settlement curve is for pier No. 3, which can be taken as typical. The comparison between calculated and observed settlements is similar to that obtained at Chelsea Bridge.

Masonic Temple, Chicago

Settlement observations have been made on a number of buildings in Chicago, but, among these, three are of outstanding value for the length and completeness of their settlement records.[25] The first to be considered here is the Masonic Temple, known later as the Capitol Building, which was a steel-frame structure completed in late 1891 and the earliest twenty-storey building in the world. The architects were Burnham and Root, and Mr E. C. Shankland was the engineer. The Masonic Temple was superseded by a more modern structure in 1939. The columns were supported on steel-grillage footings typically 20-ft square, and occupying 73% of the total plan area. They were founded at a depth of 14 ft, and under the whole building there was a basement about 10 ft deep. The

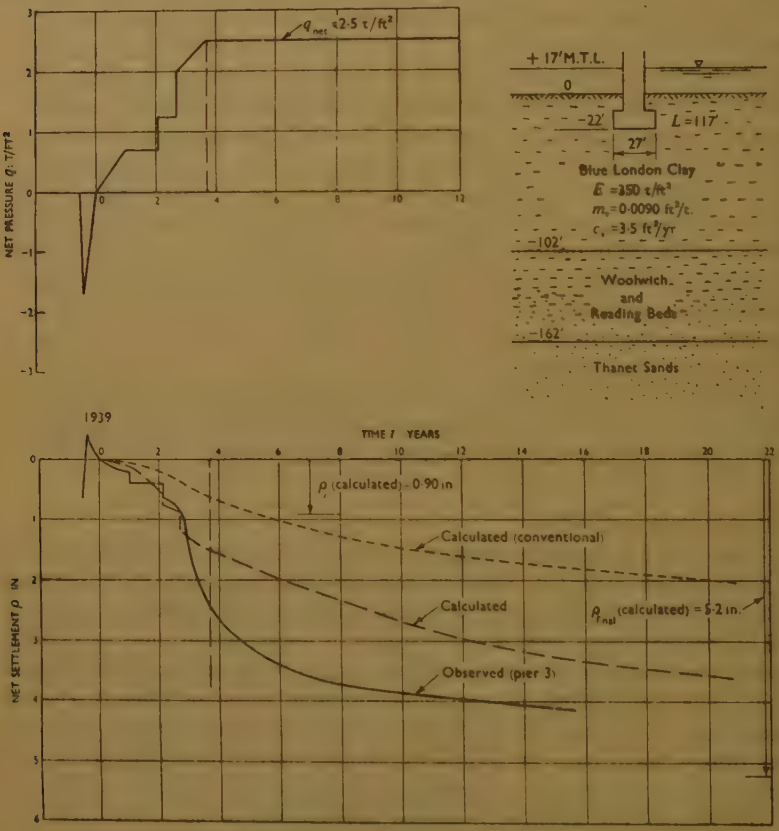

FIG. 4.—WATERLOO BRIDGE, LONDON

gross pressure under the footings was 1·38 ton/sq. ft and the corresponding net pressure was 0·67 ton/sq. ft. The footings were placed on a thin crust of stiff clay (2 to 3 ft thick) underlain by soft glacial clay, followed by a comparatively stiff glacial clay and a hard stony clay with rock at a depth of about 100 ft. Full details of the Chicago soils are given by Peck and Reed.[26]

The strength of the soft clays increases with depth and the compressibility decreases, ranging from 0·07 ft²/ton to about 0·03 ft²/ton. The compressibility is computed from the pressure/void-ratio curve corrected

TABLE 1.—SUMMARY OF DATA AND RESULTS

Structure	Date of completion of structure	Period of observation: years	Dimensions of foundation			Consolidation history of clay	Effective thickness of clay beneath foundation		Net final settlement			Net settlement at end of construction			Ratio of net settlement at end of construction to net final settlement
			B: ft	*L*: ft	*D*: ft		*Z*: ft	$\frac{Z}{B}$	Observed: in.	Calculated: in.	$\frac{\text{Calc.}}{\text{Obs.}}$	Observed: in.	Calculated: in.	$\frac{\text{Calc.}}{\text{Obs.}}$	
Fire Testing Station, Elstree . . .	1935	4	5	10	7	H.O-C	20	4·0	0·7	0·65	0·93	0·3	0·4	1·33	0·43
Chelsea Bridge, London	1937	18	28	106	31	H.O-C	60	2·0	2·1	3·3	1·57	1·2	1·0	0·83	0·57
Waterloo Bridge, London . . .	1941	16	27	117	22	H.O-C	40	1·5	3·4	5·2	1·50	1·1	1·2	1·09	0·32
Masonic Temple, Chicago	1891	22	125	182	14	N-C	49	0·4	9·8	8·8	0·90	2·3	4·0	1·74	0·24
Monadnock Block, Chicago . . .	1892	55	88	212	12	N-C	61	0·7	22·0	16·0	0·73	5·0	8·5	1·70	0·23
Auditorium Tower, Chicago . . .	1889	68	67	100	17	N-C	54	0·8	24·0	22·0	0·92	9·1	10·6	1·16	0·38

H.O-C denotes heavily over-consolidated clay
N-C denotes normally-consolidated clay

for sample disturbance in the manner described by Terzaghi and Peck.[6] The coefficient of consolidation has an average value of 45 ft^2/year and Young's modulus has an average value of 35 tons/sq. ft. The sensitivity of the soft clay is about 4.

Excavation was completed in January 1891 and settlement records commenced in May 1891 at a time when the net loading was approximately zero. Construction was completed in November 1891 and detailed settlement observations[27] were continued until September 1895. There was then a gap of about 18 years until 1913 when a complete settlement survey

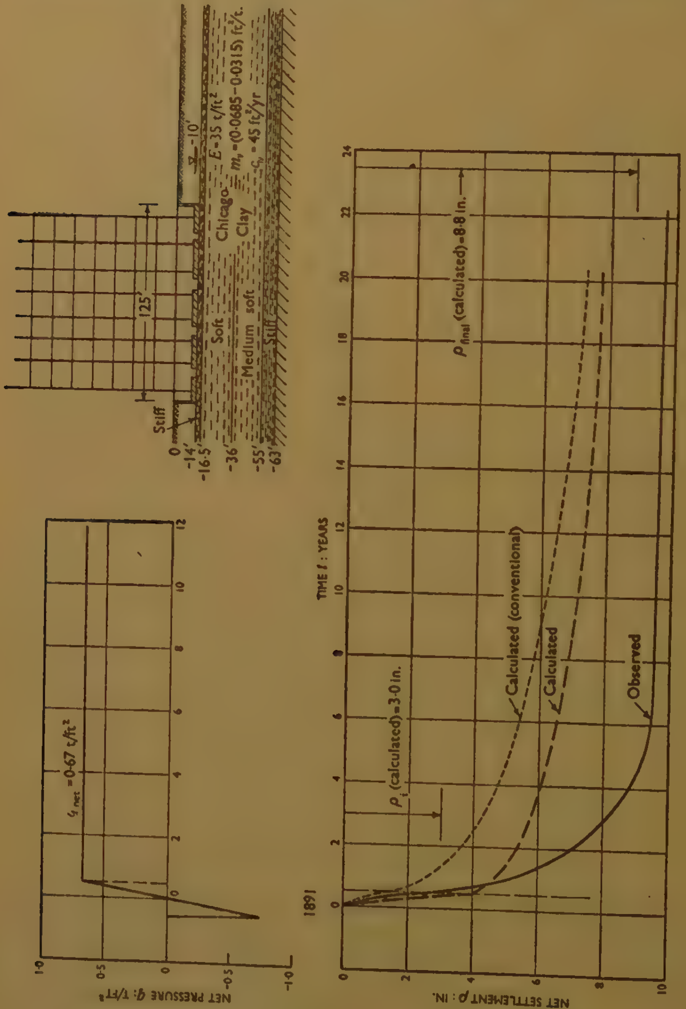

FIG. 5.—MASONIC TEMPLE, CHICAGO

was again made. The building in plan is essentially in the form of a hollow rectangle and consequently the settlements are small near the centre of the area. The average time/settlement curve for the four corners of the building is shown in Fig. 5 from which it will be seen that by 1913, 22 years after construction, settlement was virtually complete at an average value of 9·8 in. The settlement at the completion of the building was 2·3 in. Since the observations commenced at a time when the net load was approximately zero these settlements are closely equal to the net values. The gross settlements are probably 0·5 to 1·0 in. greater than these.

The footings are taken as flexible and the gross load is considered to be reduced only by the weight of soil removed immediately above each footing. The net final settlement of a corner footing, calculated from equation (1), is found to be 8·8 in.; and the net immediate settlement, from equation (2), is found to be 3·0 in. Since three-dimensional theory has not been evaluated for the ratio of foundation width and clay thickness existing at this building, one-dimensional theory has had to be used. To apply this theory to the present case, where the compressibility decreases appreciably with depth, time/settlement curves have been calculated for a number of layers of increasing thickness and the envelope of these curves has been taken as the required time/settlement relation for the building (see Fig. 5).

From these calculations the net final settlement of 8·8 in. is 1 in. less than the actual value, and the net settlement at the end of construction is 4 in. as compared with the actual settlement of 2·3 in. The calculated rate of settlement is rather less than that observed.

In Fig. 5 the conventional time/settlement curve (from equation (4)) is also given and, as usual, it considerably underestimates the settlements occurring in the years following construction.

The Monadnock Block, Chicago

The Monadnock Block was erected between 1890 and 1892, the architects being Burnham and Root. The superstructure is a load-bearing-wall building of seventeen storeys; a height which has probably never been exceeded by any structure of this type. The interior steel frame and the exterior bearing walls are supported by steel-grillage footings[28] which occupy 80% of the gross area enclosed by the exterior footing edges. The footings rest on a crust of stiff clay (4 to 5 ft thick) at a depth of 12·5 ft below the ground surface. The gross pressure beneath the footings is 1·74 ton/sq. ft and this is reduced by the excavation to a net pressure of 1·10 ton/sq. ft. Below the stiff clay crust there is a deposit of about 38 ft of soft to medium-soft Chicago clay and 18 ft of stiffer clay. These in turn are underlain by hard glacial clays and sands, with rock at a depth of about 100 ft.

The compressibility of the clay decreases with depth; the average values for the soft, medium soft, and stiff clays being respectively about

0·05, 0·03, and 0·02 ft²/ton. The average value of the coefficient of consolidation is about 45 ft²/year, and Young's modulus is approximately 35 tons/sq. ft.

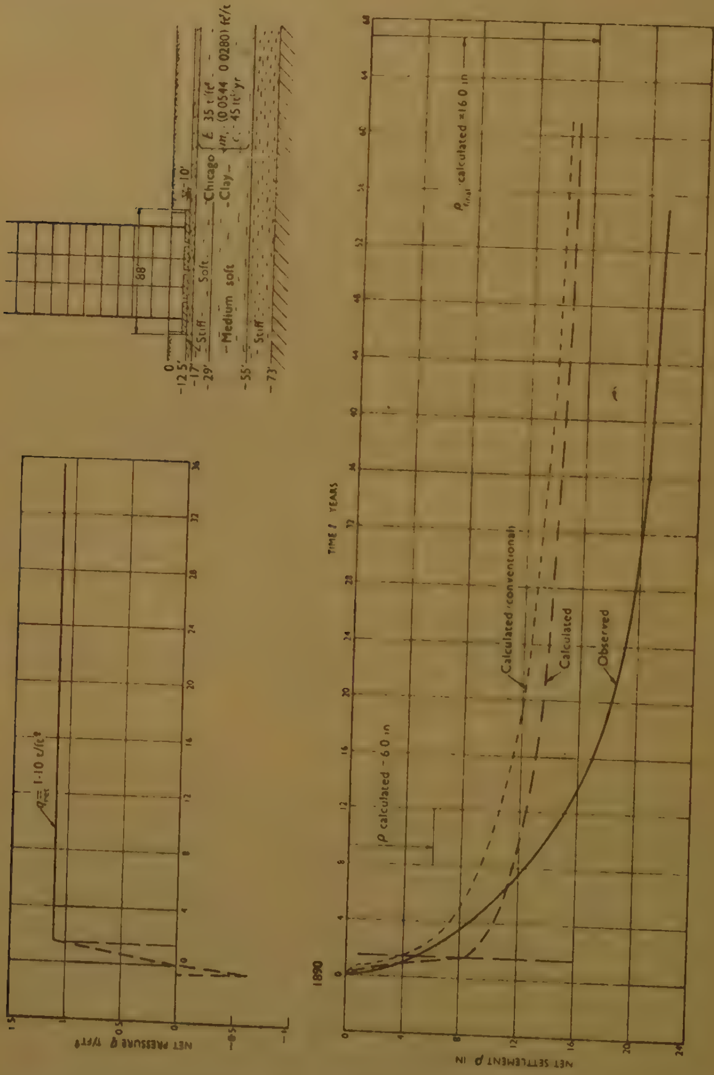

FIG. 6.—MONADNOCK BLOCK, CHICAGO

Settlement observations were begun in 1890 at the time when construction had reached ground level and the net load on the soil was rather less than zero. Observations were continued at a limited number of points in the building from that time until 1944, and the time/settlement curve for the north-east corner is shown in Fig. 6. It will be seen that about 40 years after the completion of construction the settlement had almost ceased, and the net final settlement is 22 in. At the end of construction the settlement was about 5·0 in. Since the observations commenced when the net load was approximately zero these settlements represent the net values; the gross settlements are probably 1 in. greater than these.

Calculations made in the usual manner for the settlement of the mid-point of the north-east-corner footing, assuming that the footing is flexible, gave immediate and final values of 6 in. and 16 in. respectively. The time/settlement curves computed from equation (5) and by the conventional method are plotted in Fig. 6. The calculated settlement at the end of construction is 8·5 in.

Auditorium, Chicago

Work on this combined theatre, hotel, and office building, for which Adler and Sullivan were the architects, commenced in 1887 and construction was completed late in 1889. It consists, in general, of a ten-storey building with a 19-storey tower; the theatre occupies the central part adjoining the tower (see cross-section in Fig. 7). The superstructure, like that of the Monadnock Block, is constructed with load-bearing walls. These two buildings are among the last examples of large structures of this type in Chicago, where, by about 1890, the steel frame had become almost standard for multi-storey buildings. Great care was taken in the design of the foundations, especially those of the tower,[28] for which Mr Sooy Smith was the consulting engineer, and settlement observations were commenced as soon as the piers reached ground floor level. These observations have been repeated at intervals up to the present day and provide a settlement record extending over a period of 68 years (Fig. 7). This is believed to be the longest known settlement record of a building.

The footings beneath the tower form a practically rigid mat and exert a gross pressure of 1·83 ton/sq. ft, which is reduced to a net pressure of 1·14 ton/sq. ft by the single-storey basement excavation. The underlying clay has a compressibility of about 0·06 to 0·07 ft^2/ton in the upper soft zone, decreasing to about 0·03 ft^2/ton at depths of about 60 ft below the surface. The same values of the coefficient of consolidation and Young's modulus have been taken as for the clay at the other two Chicago buildings.

Owing to the size and relative complexity of the Auditorium the period of construction was unusually lengthy, amounting to 30 months and by the end of construction the average settlement of the tower was 9 in. In another 5 years the settlement had increased to about 18 in., whilst by 1940, about 50 years after construction the settlement was virtually complete, at an average value * for the tower of about 24 in. (see Fig. 7).

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* This is the average of the four corners and the centre of the tower, corrected by a small amount to allow for the effect of a reduction in ground-water level which occurred about 1922.

Settlement calculations, made exactly as for the two other Chicago buildings (except that the foundation of the tower is assumed to be rigid),

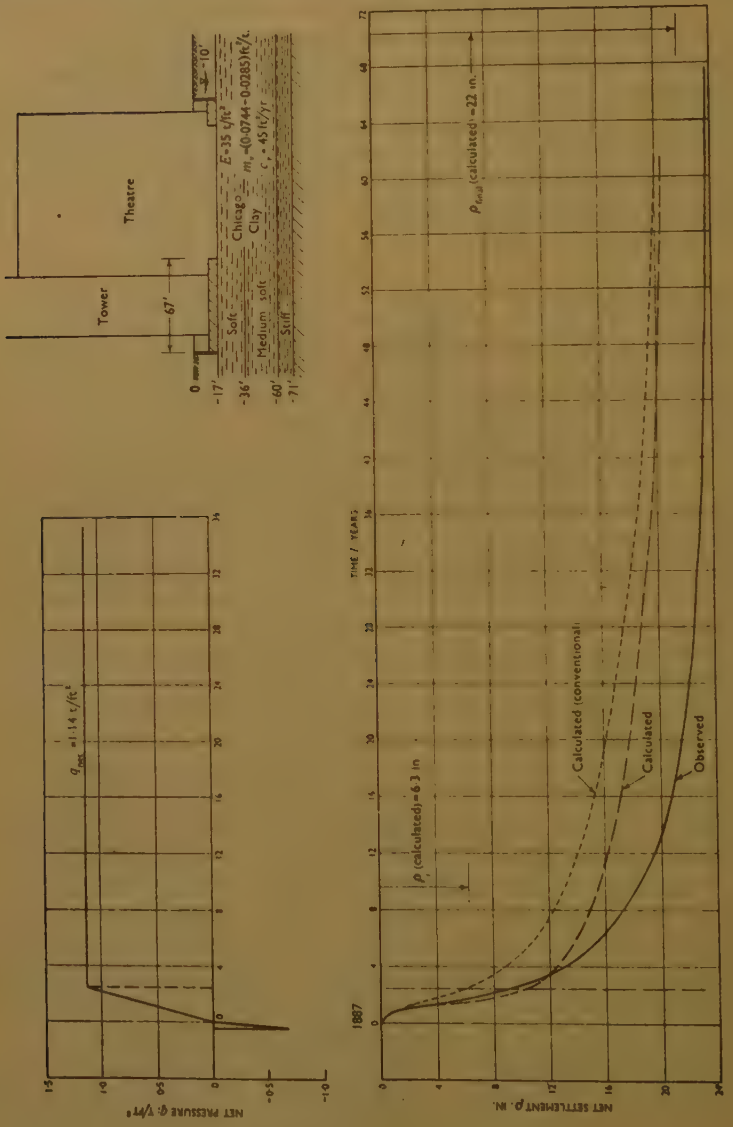

FIG. 7.—AUDITORIUM TOWER, CHICAGO

give the net final settlement as 22 in. and the net settlement at the end of construction as 10·6 in. The calculated time/settlement curve is plotted in Fig. 7 and is seen to agree rather well with the actual settlements. The conventional time/settlement curve is also plotted.

DISCUSSION OF RESULTS

The observed and calculated net settlements are summarized in Table 1 and values for the ratio of calculated to observed settlements, averaged for Chicago and London, are given in Table 2.

TABLE 2.—AVERAGE VALUES OF THE RATIO OF CALCULATED TO OBSERVED NET SETTLEMENTS FOR THREE STRUCTURES IN LONDON AND THREE IN CHICAGO

Location and type of clay	$\frac{\text{Calculated net settlement}}{\text{Observed net settlement}}$	
	Final	End of construction
London Over-consolidated clay	1·33	1·08
Chicago Normally consolidated clay	0·85	1·53
Mean for all six structures	1·09	1·31

In Table 2 the mean ratios for all six structures are also quoted, and it will be seen that the calculated net final settlement is, on the average, only 9% greater than the observed value.

As mentioned earlier, this result is closely similar to that obtained from a survey of twenty settlement comparisons.[10] The calculated net final settlements on the over-consolidated clay in London are, however, appreciably greater than the observed values. This clay is fissured and it is possible that during sampling the fissures may open slightly; thus leading to the measurement in the oedometer test of a compressibility somewhat greater than the true in-situ value. On the other hand the calculated settlements at the end of construction are in quite good agreement with the observations. Too much emphasis should not be placed on this contrast, but it is probably not irrelevant that the value of E (upon which the end-of-construction settlement chiefly depends) used for the two London bridges was determined in a recent series of tests on London Clay, whereas the compressibility of the clay at Waterloo Bridge was measured in 1937 when sampling methods were less satisfactory than at the present time.

In contrast with the London results, the calculated net final settlements on the soft normally consolidated clay in Chicago differ, on the average, by only 15% from the observed values; an agreement that is probably within the range of accuracy of the theory used in the calculations. Yet the calculated end-of-construction settlements are conspicuously greater than the observations. This is almost entirely due to the measured values of E being too low in the Chicago clay; for it is well known that in soft clays[17] this property is the most sensitive to sampling disturbance and, at present, no method of general validity has been suggested for correcting the measured values of E. In soft clays the compressibility, as determined directly from oedometer tests, is also influenced by sampling disturbance but to a smaller extent than the modulus of elasticity. The compressibility, however, can be corrected by an accepted method and, as previously stated, this correction has been applied to the Chicago test results.

Turning, lastly, to the shape of the time/settlement curves a tendency will be noted, at both Chicago and London, for the calculated rate of consolidation to be less rapid than that actually taking place. This is particularly clear at Chelsea Bridge and the Masonic Temple. If these two records alone were considered there might be a strong argument for concluding that consolidation theory required revision, that the laboratory testing was in error, or that the geological conditions were different from those assumed in the calculations. Yet the records at the Fire Testing Station and at the Auditorium show moderately good agreement between calculation and observation. It is therefore difficult to generalize on this matter, except to say that rate of consolidation seems to be the least satisfactory part of the settlement analyses of the six structures under consideration in this Paper. A comparison of the two sets of calculated time/settlement curves does show, however, that equation (5) is definitely superior to the conventional equation (4).

ACKNOWLEDGEMENTS

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The Paper, which was received on the 31st January, 1955, is accompanied by seven sheets of diagrams, from which the Figures in the text have been prepared.

CORRESPONDENCE on this Paper should be forwarded to reach the Institution by the 15th November, 1955. Contributions should not exceed about 1,200 words.—Sec.

CORRESPONDENCE
on a Paper published in
Proceedings, Part I, March 1955

Paper No. 5978

" Present Trends in the Design of Pressure Tunnels and Shafts for Underground Hydro-Electric Power Stations "†

by

Charles Jaeger, Dr ès Sc. Techn.

Correspondence

Mr F. H. Knapp, of São Paulo, Brazil, observed that the Paper constituted an outstanding review of present-day design practice for steel-lined pressure shafts and would be studied with interest by engineers engaged in that type of work. He wished to enlarge on two practical but nevertheless important problems, from the point of view both of safety and economy, namely, the stability of the lining against external water pressure and the proportioning of the steel thickness for internal pressure.

Hydrostatic pressure might be transmitted through rock fissures and the concrete pores to the lining. Engineers with experience in the construction and operation of shafts now agreed that buckling of the rather thin lining represented a major problem. The magnitude of such pressures, acting locally upon the shell, needed to be estimated on the basis of geological and hydraulic conditions. As in any engineering calculation, a certain factor of safety or, in that particular case, a factor of ignorance must be included. In most cases that pressure would be taken as the vertical distance between centre of shaft and ground-water surface.

The theory of stability of a perfectly round and infinitely long thin shell furnished the following relation between the number of waves ω in the circumference and the critical buckling stress σ_{cr} :

$$\omega^2 = 1 + \sigma_{cr} \cdot \left[\frac{r}{e}\right]^2 \cdot \frac{12 \cdot \left[1 - \frac{1}{m^2}\right]}{E} \quad . \quad . \quad . \quad . \qquad (a)$$

where m denoted Poisson's ratio for steel (10/3).

In order to make use of that equation, a relation between ω and the deflexion of the shell, restrained by the concrete envelope, was required. Denoting the length of one half wave by l, measured on the circumference with radius r, the deflexion curve was given by:

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† Proc. Instn Civ Engrs, Part I, vol. 4, p. 116 (Mar. 1955).

$$y = y_{\max} . \sin \frac{\pi . x}{l} = y_{\max} . \sin \frac{x}{r} \omega$$

Those deflexions were exceedingly small compared with the wave length, and the total length of the deflected shell amounted to:

$$L' = 2 . \omega . 2 . \left[\int_0^{l/2} dx + \tfrac{1}{2} \int_0^{l/2} \left(\frac{dy}{dx} \right)^2 . dx \right] = 4 . \omega . \left[\frac{l}{2} + \frac{\pi^2}{8} . \frac{(y_{\max})^2}{l} \right]$$

the circumferential difference of length before and after buckling to:

$$L = 4 . \omega . \left[\frac{l}{2} + \frac{\pi^2}{8} . \frac{(y_{\max})^2}{l} \right] - 2 . \omega . l = \omega . \frac{\pi^2}{2} . \frac{(y_{\max})^2}{l}$$

and, finally, the average radial and inward deflexion to:

$$y_d = \frac{L}{2 . \pi} = \frac{\omega . \pi}{4} . \frac{(y_{\max})^2}{l} = \frac{\omega^2 . (y_{\max})^2}{4 . r} \quad . \; . \; . \quad (b)$$

Assuming that the shell had been prestressed with a stress σ_{pr} by means of grouting between steel and concrete, the shell suffered a radial shortening during buckling of:

$$\frac{\sigma_{cr} - \sigma_{pr}}{E} . r \quad . \; . \; . \; . \; . \; . \quad (c)$$

With a temperature drop of the lining, the shell deflected radially by an amount equal to:

$$K . \Delta t . r \quad . \; . \; . \; . \; . \; . \; . \quad (d)$$

The condition of compatibility of the various deflexions then took the form:

$$y_{\max} = \frac{\sigma_{cr} - \sigma_{pr}}{E} . r + K . \Delta t . r + \frac{\omega^2 . (y_{\max})^2}{4 . r} \quad . \; . \quad (e)$$

The maximum stress in the shell due to direct and bending stress had a value of:

$$\sigma = \sigma_{cr} . \left[1 + \frac{6 . y_{\max}}{e} \right] = \sigma_Y \quad . \; . \; . \quad (f)$$

and could not exceed the yield point of steel.

Then, introducing ω^2 from relation (a), $y_{\max}$ from relation (f) into (e), and simplifying, the following equation resulted:

$$\left(\frac{r}{e} \right)^2 . \left[\frac{\sigma . \sigma_{cr}}{\sigma_Y - \sigma_{cr}} \left\{ \frac{\sigma_{cr} - \sigma_{pr}}{E} + K . \Delta t \right\} + \frac{\sigma_Y - \sigma_{cr}}{2 . E} \right]$$

$$- \frac{r}{e} + \frac{1}{24} \frac{\sigma_Y - \sigma_{cr}}{\sigma_{cr}} = 0 \quad . \quad (1)$$

That relation had been represented in Fig. 44 for various prestress ratios σ_{pr}/E, a drop of temperature of 5°C, and a yield point for the steel of 2,800 kg/cm². Prestressing the lining by means of pressure grouting had thus a considerable and beneficial influence upon the buckling strength of the shell.

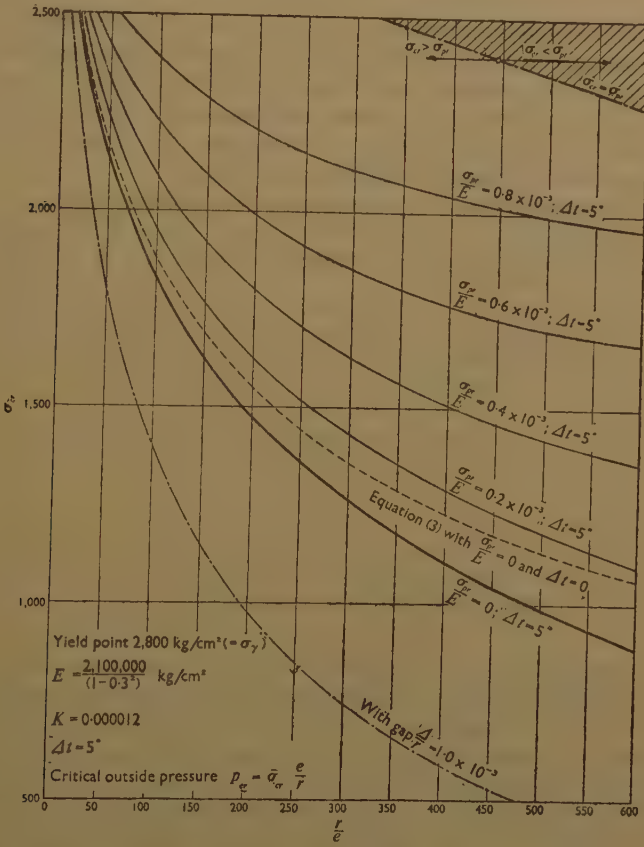

FIG. 44.—CRITICAL BUCKLING STRESS OF THIN STEEL SHELLS

The first Paper by Amstutz mentioned by the Author (and which contained an inadmissible approximation) assumed a gap Δ between steel and concrete, caused by shrinkage of the concrete. For that case, relation (*e*) became:

$$y_{\max} = \Delta + \frac{\sigma_{cr}}{E} \cdot r + \frac{\omega^2 \cdot (y_{\max})^2}{4 \cdot r}$$

and equation (1):

$$\left(\frac{r}{e}\right)^2 \cdot \left[\frac{6.\sigma_{cr}}{\sigma_Y - \sigma_{cr}}\left\{\frac{\Delta}{r} + \frac{\sigma_{cr}}{E}\right\} + \frac{\sigma_Y - \sigma_{cr}}{2.E}\right] - \frac{r}{e} + \frac{1}{24}\,\frac{\sigma_Y - \sigma_{cr}}{\sigma_{cr}} = 0\ . \qquad (2)$$

That relation, for a specific gap of $\frac{\Delta}{r}$ equal to $1{\cdot}0 \times 10^{-3}$, had also been shown in Fig. 44 and indicated clearly the decrease of the buckling strength and thus the necessity of avoiding any gap between steel and concrete. Mr Knapp did not recognize the validity of the assumption of concrete shrinkage in a pressure shaft, which took place only when subject to a drying process. On the contrary, it might even be expected that the concrete would swell owing to the presence of water.

Neglecting any prestressing of the shell and considering that in equation (1) the member $\frac{\sigma_Y - \sigma_{cr}}{2.E}$ might be neglected compared with $\frac{6.\sigma_{cr}^2}{E.(\sigma_Y - \sigma_{cr})}$, also the member $\frac{\sigma_Y - \sigma_{cr}}{24.\sigma_{cr}}$ in relation to $\frac{r}{e}$, that equation assumed the form:

$$\left(\frac{r}{e}\right)^2 \cdot \frac{6.\sigma_{cr}}{\sigma_Y - \sigma_{cr}} \cdot \frac{\sigma_{cr}}{E} - \frac{r}{e} = 0 \quad . \ . \ . \ . \ . \ . \qquad (3)$$

and with

$$p_{cr} = \sigma_{cr} \cdot \frac{r}{e}$$

$$p_{cr} = \frac{E}{12} \cdot \left(\frac{e}{r}\right)^2 \cdot \left[\sqrt{1 + 24 \cdot \frac{\sigma_Y}{E} \cdot \frac{r}{e}} - 1\right] \quad . \ . \ . \qquad (4)$$

Equation (3) had been represented in Fig. 44, the differences between that form and equation (1) being negligible for very small ratios of r/e.

The validity of equations (2), (3), and (4) needed to be judged on the basis of its derivation and Mr Knapp felt that the assumptions made were in accordance with established facts. Out-of-roundness of the shell, however, had been neglected on the ground that linings for pressure shafts compared with smaller penstocks were fabricated within closer tolerances.

So far as the proportioning of the thickness against internal pressure was concerned, it seemed reasonable to assume that up to a certain pressure the rock would be able to support tension stresses without cracking.

With an inside pressure p_i the amount $\epsilon \, . \, p_i$ would be taken up by concrete and rock. With a prestress of $\lambda \, . \, p_i$ the lining carried a pressure of:

$$p_i - \lambda \, . \, p_i - \epsilon \, . \, p_i = p_i \, . \, [1 - \lambda - \epsilon]$$

Denoting the outside radius of the concrete by r_a and the radius to the unfissured rock by R, the radial shortening of the cracked concrete and rock amounted to:

$$\Delta_1 = (\lambda + \epsilon) \cdot p_i \cdot r_i \cdot \left[\frac{1}{E_c} \cdot \log_e \frac{r_a}{r_i} + \frac{1}{E_r} \cdot \log_e \frac{R}{r_a}\right]$$

where E_c and E_r represented the modulus of elasticity of concrete and rock.

The radial extension of the sound rock of theoretically infinite thickness had a value of :

$$\Delta_2 = \frac{1}{E_r} \cdot \frac{m+1}{m} \cdot [\lambda + \epsilon] \cdot p_i \cdot r_i$$

with m, Poisson's ratio for rock, equal to about 6.

The extension of the shell was given by :

$$\Delta_3 = \frac{r_i^2}{E \cdot e} \cdot p_i \cdot [1 - \lambda - \epsilon]$$

The condition of compatibility of deformations required that :

$$\Delta_2 = \Delta_3 - \Delta_1$$

Introducing the values, and simplifying, gave :

$$\epsilon = \frac{1}{1 + \frac{E}{E_c} \cdot \frac{e}{r_i} \cdot \log_e \frac{r_a}{r_i} + \frac{E}{E_r} \cdot \frac{e}{r_i} \cdot \left[\log_e \frac{R}{r_a} + \frac{m+1}{m}\right]} - \lambda \quad . \quad (g)$$

The steel received a circumferential stress of :

$$\sigma = \frac{p_i \cdot (1 - \lambda - \epsilon)}{e} \cdot r_i \quad . \; . \; . \; . \; . \; . \quad (h)$$

Substituting the steel thickness from (h) into (g) :

$$\epsilon = \frac{\sigma}{p_i} \cdot \frac{1}{\frac{E}{E_c} \cdot \log_e \frac{r_a}{r_i} + \frac{E}{E_r}\left[\log_e \frac{R}{r_a} + \frac{m+1}{m}\right]} - \lambda \quad . \quad (i)$$

In that relation the radius R, separating the cracked from the sound rock, was unknown. However, according to Lamé's equation for an infinitely thick cylinder, the tangential and radial stresses in any point of the rock were equal but of opposite sign, so that :

$$\sigma_t = p_i \cdot [\lambda + \epsilon] \cdot \frac{r_i}{R} \cdot \left(\frac{R}{R}\right)^2 = p_i \cdot [\lambda + \epsilon] \cdot \frac{r_i}{R}$$

Thus, the radius R amounted to :

$$R = [\lambda + \epsilon] \cdot \frac{p_i \cdot r_i}{\sigma_t} \quad . \; . \; . \; . \; . \; . \quad (5)$$

From equation (i) it followed that :

$$E_r' = E_r \cdot \frac{\frac{m+1}{m}}{\frac{m+1}{m} + \log_e \frac{R}{r_a}} \quad . \; . \; . \; . \; . \quad (6)$$

where E_r' represented the " apparent " modulus of elasticity of rock. That relation indicated a decrease of the modulus with increasing pressure because the cracks spread farther into the mass of rock.

Whether it was possible to ascribe a constant tensile strength to sound rock surrounding a pressure shaft remained a hypothesis. It was, therefore, of the utmost importance to compare the theoretical derivations with experiments.

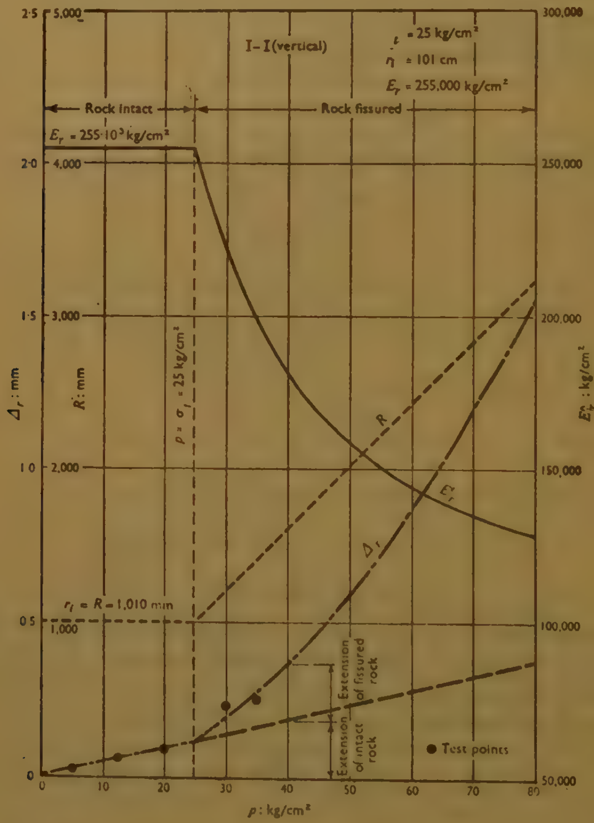

FIG. 45.– COMPARISON OF TEST RESULTS WITH COMPUTATION

The results of such tests, carried out in a test chamber with a diameter of about 2 m, excavated in sound granitic gneiss of excellent quality were shown in Fig. 45. Computed and measured extensions of the rock agreed fairly well. Those tests in vertical and horizontal planes of two measuring sections revealed the fact that the deformations in the vertical planes were always greater than those in the horizontal ones. That difference might be explained by the dip and strike of the rock formation. In the particular case of those tests, the maximum pressure applied was limited by the capacity of the pump. The fact that test and computation agreed offered the possibility of computing the apparent modulus of elasticity for higher pressures. Those results did not contradict the Author's statement that tensile stresses developing in the rock seemed to have no effect on the stability of the lining and that fissured rock remained solid.

Equation (1) might then be written :

$$\epsilon = \frac{\sigma}{p_i} \cdot \frac{1}{\frac{E}{E_c} \cdot \log_e \frac{r_a}{r_i} + \frac{E}{E_r{}'} \cdot \frac{m+1}{m}} - \lambda \quad . \quad . \quad . \quad (7)$$

Mr Knapp's ideas in regard to design and construction of pressure shafts might be summarized as followed :—

(1) The lining should be designed against external pressure without any consideration of prestressing or gap.

(2) Measurements should be carried out in a test chamber to ascertain the apparent modulus of elasticity of the rock.

(3) The constant load carried by the rock should be determined and an estimate should be made of the appropriate amount of prestress to be produced by grouting between steel and concrete and between concrete and rock.

(4) The shell thicknesses should be determined when they exceeded those required for external pressure.

(5) After completion the shaft should be subjected to a pressure test. During that test, concrete and rock would suffer a permanent set, thus creating a gap between steel and concrete.

(6) The gap should be closed by pressure grouting, using internal strutting by means of collapsible stiffener-rings mounted on a movable carriage as an additional safeguard against local buckling.

A pressure shaft designed along such lines would transmit most of the load to the rock. Critical conditions occurred after the pressure test and before or during grouting the lining, as shown by many known partial failures caused by local buckling.

So far as drains were concerned, Mr Knapp considered that such permanent drains were not only dangerous due to plugging (in Brazil the water was extremely soft, and, therefore, aggressive) but useless to drain the surrounding rock. Drains provided to relieve the pressure between shell and concrete possessed the same disadvantage and served no useful purpose, provided the steel lining had been designed against external pressure together with a carefully executed grouting programme after the pressure test. Such a procedure made external anchorages superfluous which was a nuisance for an efficient concreting job. Usually such anchorages were not elastic enough and in one case the lining had buckled locally, carrying with it not only the anchors but large chunks of concrete.

The Author had not mentioned one controversial subject—the gradient of pressure shafts. Swiss authorities held that an inclination between 65 and 75% gave most efficient and safe working conditions. Slopes of 90% or more were required for chuting of the excavated rock, but such inclinations were hazardous. The Limberg shaft of the Glockner-Kaprun development in Austria set the record with a gradient of 125%, although in that particular case movable storage bins and cars had been used to remove the spoil.

The studies which had been outlined were undertaken in connexion with the design of two large underground plants, Nilo Peçanha (formerly Forçacava) located near Rio de Janeiro and Cubatão in the vicinity of the port of Santos; the principal data of both developments were given in Table 9.

Mr Knapp was indebted to "Companhia Brasileira de Administração e Serviços Tecnicos" (Cobast—Light and Power) for permission to publish test results and data. The carefully conducted tests in the pressure chamber for the Cubatão underground plant were carried out by the "Instituto de Pesquizas Tecnologicas" of São Paulo, Brazil.

Mr J. F. Hagrup, of Stockholm, observed that the steel and concrete lining of penstocks and tunnels for an underground power station introduced numerous problems which had been discussed in detail by the Author. From a theoretical point of view it was natural to assume that a part of the internal water pressure was transferred from the steel lining to the concrete and the rock, which would permit an appreciable reduction in the thickness of the plate. When making the calculations for the penstocks in the underground power stations constructed by the Swedish State Power Board which in many instances amounted to 5 to 8 m in diameter it was assumed that both the internal and external water pressure were taken up solely by the steel lining. The reasons for that were as follows.

The equations for calculating the partial water pressure transferred from the steel lining to the rock included the modulus of elasticity of the rock. Since rock was not a homogeneous material the modulus of elasticity might vary within wide limits and its value was difficult to determine. On that account the results of the calculations might lead to incorrect conclusions.

To enable a part of the pressure to be transferred from the steel lining to the rock, complete contact must be established between the plate and the intermediate concrete. If the concrete was placed in the customary manner it was impossible to prevent shrinkage, even when the concrete was vibrated. Consequently, a small gap was formed between the steel lining

TABLE 9

	Units :	Nilo Peçanha	Cubatão
General			
Gross static head (max.)	metres	343·5	720·4
Discharge (max.)	cubic metres per second	125·0	75·0
Capacity supplied by one shaft (max.)	kilowatts	355,000	465,000
Date of commission		1953/54	Under construction
Pressure shaft			
Rock		Gneiss with kaolinized zones	Granitic gneiss
Gradient	per cent	90	90
Diameter	metres	6·10	3·25
Ultimate strength of steel (min.)	kilograms per square centimetre	4,950	5,300
Yield point of steel (min.)	,,	2,950	2,950
Thickness at bottom (max.)	millimetres	17·4	23·0
Stress without rock support	kilograms per square centimetre	6,050	5,050
Power house			
Number of units		6	6
Type of units		Reaction vertical	Impulse, 4 sets vertical
Distance between units	metres	16·0	15·6
Position of transformers		Opposite units	Opposite units
Crane capacity	tons	2 × 125	2 × 100
Crane span	metres	12·8	19·8
Total height	,,	32·4	39·7
Total width	,,	18·4	19·0
Total length	,,	114·2	120·0
Volume of power house	cubic metres	75,000	94,000
Volume of valve chamber (opposite power house)	,,	—	6,600
Volume per kilowatt	cubic metres per kilowatt	0·21	0·22

and the concrete, so that only an insignificant part of the pressure was transmitted to the rock. The special methods suggested by the Author for the elimination of the gap, such as grouting and prestressing, should of course be considered if they were found to yield better economic results.

There was always a certain risk that the concrete between the steel lining and the rock might be subjected to a gradual destruction. That applied in particular to the concrete nearest to the rock, which came into direct contact with the ground-water. The concrete thereby lost its sustaining capacity, which might lead to such extensive deformation in the plate that the breaking point of the latter was exceeded. Since it was very difficult to get at the concrete for repairing the damage in some Swedish power stations where the rock was of very sound quality, the steel lining had been omitted and the concrete was reinforced instead. The question had also been discussed in such cases as to whether or not it would be possible to omit both the steel and concrete lining and merely smooth the surface of the rock by guniting. To prevent damage to the latter when emptying the penstocks the rock must be drained by means of vertical drill-holes.

The concrete between the steel lining and the rock was not provided with drainage since such drainage was usually expensive and complicated. Moreover, there was a risk that that form of drainage would gradually become clogged. Thus, as already mentioned, when making the calculations for the steel lining the external water pressure must also be taken into account since that was usually the determining factor for the thickness of the steel lining. In order to prevent inward buckling of the plate it was generally found economical to increase the moment of inertia by the provision of stiffening ribs, which were welded round the piping at given intervals. On the vertical section of the latter the ribs were set at a slight upward angle in relation to the plate for the purpose of preventing, so far as possible, the formation of hollow spaces in the concrete under the ribs.

Mr Jean Pelletier, of Paris, referring to " steel-lined pressure shafts " mentioned by the Author, said he would like to add to the information given in Table 5.

The Cordéac's inclined shafts, which were double, were quite unusual in design for at least two reasons. The shafts were not in fact steel lined but were lined with reinforced concrete. Only a few metres near the turbine had a steel lining.

In addition, the shafts were of the Gerlos type described by the Author as having caused the worst failures in underground pressure shafts.

The main difference in the Gerlos type was that the inspection gallery was located above the pressure pipe.

Built during the 1939–45 war under regulations of the German authorities the use of a steel penstock was not allowed, the reason given being the shortage of steel and the necessity to save it.

The designing bureau studied a scheme for a steel-lined underground shaft, but the scheme was rejected as being uneconomical in steel consumption. The next proposal was for an inclined reinforced concrete lined shaft using welded hoops as reinforced bars, which was accepted by the Germans. However, when the designs were completed, it was found that the weight of the steel bars was more than the steel sheet which would have been required for the steel-lined type.

Nevertheless, that type of shaft was used since it was the only method allowed. It was very difficult work setting the bars; the problem was how to pour the concrete through them. It was accomplished by a concrete pump and vibrators; the shutters which were used were home-made wooden ones.

The Piage-de-Vizille shaft was designed and built in the years immediately following the end of the 1939–45 war. At least, in that case, the steel-lined type of shaft was permitted. The concrete between the sheet and the rock was inserted by means of an air placer and grouted in twice; the first grouting was for filling the voids and the second at high pressure for blocking to the sound rock.

The third shaft to be considered, that at Brevières, was much more interesting. It was a modified Gerlos type as used at Cordéac, but steel lined.

The general features of the inclined tunnel at Brevières were shown in Fig. 46.

The construction could be summarized as followed. The tunnel was located in metamorphic rock, half in quartzite and the remainder in schist. After excavation to the design shape had been carried out and carefully checked, the concrete for the invert was poured enclosing a rail-track, the top of the rail just emerging from the concrete. Steel pipes, strengthened and stiffened by inside guys, from 6 to 8 m long, were hauled on dollies from the downstream entrance of the tunnel and were settled in place on light walls rising from the invert half way up the pipe cross-section. The guys were used in two pipes during welding operations and afterwards removed upstream to another part. After the pipes had all been welded together and the guys removed, concrete was poured almost to the top of the pipe using a concrete pump.

Since the tunnel was 900 m long (990 yd) two pumps were used, the first pouring into the hopper of the second. Concrete was made in a central mixing plant outside the tunnel and hauled to the pump site near the tunnel entrance. It was improved in a special remixer before being poured into the hopper of the first pump.

The reinforcing bars were placed before pouring the second phase using nearly the same plant, and the third operation for completing the inspection gallery was poured in the same way using light prefabricated concrete shutters, which were abandoned in the concrete. Grouting operations were carefully made in two stages; first, the voids were packed so that there was a good bond between the steel and the concrete and, secondly, high-pressure grouting was used to block the contact to the sound rock.

The height of the visiting gallery was only 1 m 25 cm (4 ft), so that men had to stoop inside. Since that would be very tiring for a distance of 900 m (990 yd) four rooms were provided for rest. Those were spaced at 180-m intervals (196 yd) and had a height of 2 m (6 ft). Reinforcement was of course heavier at those sites, in order to improve the strength of the concrete in those unsymmetrical and higher sections.

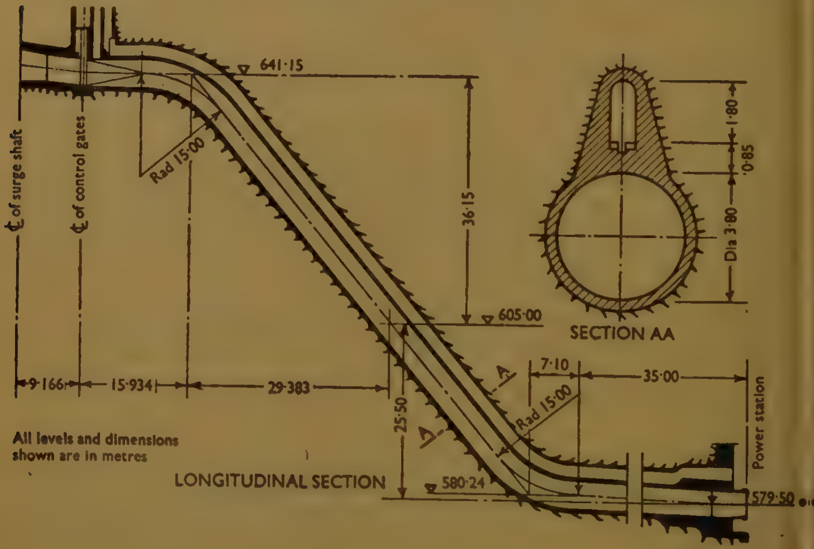

Fig. 46.—Pressure tunnel 3·80 metres in diameter

Drainage holes were provided every 50 m (54 yd) on each side of the visiting gallery. Since those holes became packed by the grouting operations, it was necessary to redrill them afterwards.

Mr C. G. Chatham noted Dr Jaeger's reference to the use of the "Prepakt" method in the penstocks at Kemano in British Columbia. At Kemano the penstocks were embedded in three monolithic sections. The first was 400 ft long and, during the course of the embedment, the operators became familiar with the simple requirements of the intrusion Prepakt technique. The procedure worked so smoothly that the remaining 800 ft of the upper section was embedded in a single operation. Thus encouraged, the 2,000 ft of penstock from the power house to level 1,600 O.D. was placed and welded. Aggregate emplacement was then started through a 10-in.-dia. pipe and when the filling had proceeded to about one-third of the way along, mortar pumping was started and both operations went on simultaneously. The intrusion-mortar pumping was continuous so that the whole 2,000 ft was encased monolithically.

The mortar pumping was at such a rate that the fluid zone was about 20 ft deep at all times; thus, the steel liner plate of the penstock was prestressed in compression by a fluid pressure of about 18 lb/sq. in. That was sufficient to ensure positive contact between the steel and the concrete after the concrete had lost its heat of hydration and cold water had been introduced into the penstock.

The turbine-pit liners and the Pelton-wheel cases in the underground power house were also embedded in Prepakt concrete.

Concerning the process itself, the coarse aggregate could range from a minimum size of $\frac{3}{4}$ in. (i.e., retained on a $\frac{3}{4}$-in.-mesh sieve) up to any maximum size which was economically available and convenient to handle, or as limited by the spacing of reinforcement and formwork. The shape of the coarse aggregate particles, their possible segregation together with minor hang-ups around reinforcement and consequent voids were not harmful, however, because all such voids would be completely filled by the intrusion mortar which, when hard, was fully as sound and strong as the concrete. In horizontal penstocks the coarse aggregate could be placed by a short length of conveyor or by blowing with low-pressure air, and for open mass-work emplacement the use of a grab was quite satisfactory.

After the coarse aggregate had been placed in the forms intrusion mortar was pumped into the voids, either through the forms or through pipes placed within the aggregate. In addition to Portland cement, sand, and water, intrusion mortar contained "Alfesil" and intrusion aid. Alfesil was the trade name for a finely divided siliceous material which replaced from 30% to more than 50% of the cement which would otherwise be required, and which reacted with the lime liberated by the cement during setting to form insoluble strength-producing compounds. The lime-Alfesil reaction also greatly reduced permeability at later ages, making the concrete 5 times as impermeable as ordinary concrete. The Alfesil particles separated the cement grains in the fluid mortar, delayed the initial set and, being spherical in shape, improved the flow characteristics of the mortar.

The intrusion aid, used in very small quantities, gave the mortar the properties of colloidal suspension and stability, simplified greatly the mixing operation, eliminated completely the effects of settling and setting shrinkage, lowered the water requirement, delayed early stiffening of the mortar and provided, in the hardened concrete, the weather-resisting characteristics of the best air-entrained conventional concrete.

The process was particularly suitable for underwater work. Intrusion mortar pumped into voids of pre-placed aggregate under water would displace all free water completely without any dilution whatever. It was therefore, neither necessary nor desirable to increase the cement content for Prepakt concrete produced under submerged conditions. The intrusion mortar pumped under water would show no laitance or lack of consistency at the water contact surface. Cold joints formed within submerged aggregate by interruptions to the pumping operation of several days' duration were therefore clean, requiring no surface treatment. Furthermore, the partial embedment of aggregate particles in the hardened portion provided a perfectly keyed joint. In fact, so excellent was that key that when cored cylinders were taken through such a joint it was almost impossible to detect its presence.

Prepakt concrete with 280 to 375 lb. of cement plus 112 to 150 lb. of Alfesil per cu. yd normally showed a strength of more than 3,500 lb/sq. in. at 28 days, and 5,000 to 7,000 lb/sq. in. at 90 days. The rate of strength gain in Prepakt concrete was slower than for conventional concrete but continued for a much longer period. It was suggested, therefore, that in mass concrete where high early strength was not required, the mix should be designed for a required strength at 90 days instead of 28 days. Past experience, as indicated by cores taken from the scroll-case embedment concrete at the Bull Shoals power house, showed that a strength of 3,000 to 4,000 lb/sq. in. was easily obtainable in 90 days with 224 lb. of cement plus 224 lb. of Alfesil per cu. yd and 43% voids in the coarse aggregate. The above void content was high, incidentally, due to the difficulty of placing aggregate in the trapped areas under the scroll cases. Normal void content for 3-in.-maximum-size aggregate was 40%. With a larger maximum size plus the more open placing conditions which often existed in mass work, the void content should approach 35% with a consequent reduction of cement plus Alfesil content to 184 lb. each.

To sum up, the special properties of Prepakt concrete were :—

(1) Any desired ultimate strength.
Normal range : 2,500 to 4,000 lb/sq. in. at 28 days.
3,000 to 7,000 lb/sq. in. at 90 days.
Maximum range : 500 to 13,000 lb/sq. in. at 365 days.

(2) Low cement content.
Normal range : 200 to 400 lb/cu. yd.
Minimum range : less than 100 lb/cu. yd.

(3) Low temperature rise : 20 to 60% less than conventional concrete.

(4) No setting shrinkage because of the inclusion of intrusion aid in the mortar.

(5) Because of the point-to-point contact of the pre-placed coarse aggregate there was 50% less drying shrinkage.

(6) High permanent bond to existing concrete. More than 80% efficiency at joints of Prepakt to prepared old concrete. The corresponding joint efficiency of new conventional concrete to old concrete was less than 50%.

(7) Excellent weather resistance.

(8) High impermeability. Structural grade was approximately five times more impermeable at the age of 6 months compared with conventional concrete. In lean mass concrete (200 lb. cement/cu. yd) it was at least equal to the best conventional mass concrete used in dams.

(9) Superior resistance to action of salt water.

(10) Underwater placement was a special feature since the intrusion mortar displaced water from the voids in the aggregate without dilution.

(11) The ratio of water to cement + Alfesil ranged from 0·40 to 0·47 for structural-grade mixes and from 0·45 to 0·58 for lean mass-concrete mixes.

Applications for hydro-electric projects included :—

(*a*) Construction of dams where only one-third of the volume of the materials had to pass through the mixer.

(*b*) Grouting of foundations.

(*c*) The aggregate grouting operation in tunnel and shaft linings also stabilized weak and permeable areas by lateral penetration.

(*d*) Diversion-tunnel plugs.

(*e*) Mass-concrete foundations in power houses.

(*f*) Embedment of turbine scroll-casings.

(*g*) Heavily reinforced and complicated structures where placement of conventional concrete became a difficult operation.

(*h*) Soil stabilization.

(*i*) Core walls in sands and gravels by a drilling, jetting, and intrusion-grouting procedure for placing grouted cut-offs in those formations. That was particularly useful for stopping flows under dams and cofferdams where boulders made the driving of sheet-piling impossible. There was no depth limit.

(*j*) Blanket grouting where the foundation material was exceptionally porous as in coral limestone or a cavernous limestone formation ; an intrusion-grout blanket might be placed under and around the proposed foundation prior to excavation to hold out the water. When caverns were encountered sand might be used in intrusion grout in place of much of the cement. A mix which consisted of 12 cu. ft of sand per

100 lb. of cement and Alfesil had been mixed and pumped into underground voids.

(*k*) Repairs to concrete-lined tunnels damaged by the effects of cavitation and many other repair jobs. Prepakt concrete had been used in that work for more than 15 years.

The above examples covered the field of hydro-electric and power-station construction but there were many applications in other fields, particularly in bridge-pier construction where a recent record for aggregate placement reached 2,000 tons/hour and 24,780 cu. yd of intrusion-Prepakt concrete was made in 7 days. It was indeed a very versatile process with great possibilities.

Mr Joseph Talobre, of Paris, thought that Dr Jaeger's Paper would be very helpful in clarifying the basic problems of pressure-shaft design. He specially noticed the place Dr Jaeger gave to the hydrostatic theory of Heim, about natural stress distribution in the rock.

Whereas in France and Switzerland the Heim rule was generally adopted, in Germany and Great Britain many engineers were rather in favour of the Terzaghi theory, which involved the uniform distribution of rock stresses along horizontal planes, with vertical predominant compressions. He considered that those engineers were right so far as soil mechanics was concerned, but that no convincing evidence appeared for application of the Terzaghi theory to rocks.

As a matter of fact, very deep rocks rarely presented horizontal beddings. Metamorphism, slaty cleavages, joints, and faults had modified the homogeneous structure of the ground. In mountainous zones, microseisms and creep continuously affected the equilibrium of former strata. The general tendency was obviously towards smaller shearing stresses, thus making, with duration, the approach to the Heim rule more accurately.

It would not, of course, be asserted that the Heim law was unconditionally reliable. In many formations, however, the Heim rule had agreed tolerably well with measurements. Such was the case in London Clay, as was shown by Dr L. F. Cooling at the 1954 Zurich Congress.

That was also the case in Bearpaw shale, as shown by the Casagrande experiments described by Mr Peterson. In 1951, an explanatory square-section drift was excavated in that shale, on the bank of the South Saskatchewan River. The drift was lined with forty reinforced concrete slabs supported by forty special struts. The pressure on the concrete slabs was originally given the value corresponding to 1·3 times the weight of the 64 ft of overburden. Two years later, a gauging demonstrated that the horizontal pressure was unchanged, but that the vertical pressure had decreased to the precise overburden value. Consequently, the vertical compression appeared to be the lowest. In deep Electricité de France tunnels, numerous surveys in schist, gneiss, and granite had never substantiated any prevailing direction of pressure or any vertical predominance of efforts. Therefore, Mr Talobre believed that adoption of the Heim rule constituted the best insurance against the hazards of stress magnitudes and stress directions in deep rocks.

At any rate, the Terzaghi pattern would have to be discarded for all practical purposes. The excavation equalized the radial stresses, since those were uniformly zero at the wall. By application of the Terzaghi law, Dr Kastner had calculated that plastic zones would arise at the sides of galleries, thus correcting unevenness in the wall stresses. In pressure shafts, inner pressure was hydrostatic. That meant that the lining must be a structure of equal strength, i.e., a radially symmetrical structure. If the rock was to act as a support to the construction, the intensity of that support must be the same at every point on the contour. The engineer would frequently have to create artificially, radially symmetrical conditions in the rock, for instance, when grouting and prestressing. If, for a good design, rock anisotropy was still to be considered, former discrepancies in the Heim rule should in fact be neglected in construction problems.

It would be seen that determination of rock characteristics was uncertain, even with the help of the most accurate admeasurements. In tunnel problems, a search for absolute accuracy would accordingly be quite unjustified.

The Heim theory, associated with the overburden thumb-rule, could give in all cases a sufficient evaluation of mean rock pressures. It answered perfectly the needs of tunnel engineers.

Mr A. Terrisse, of Geneva, said he wished to supplement the information given on pp. 125, 126, 143, 144, 150, and 151 concerning the lined shaft of the Gondo power station.

Referring to p. 125 the addition of 50 kg of pozzolanic cement per cu. m of concrete (3·13 lb/cu. ft) had been employed only in the concreting of the part of the shaft not lined with steel. Pozzolana was used to settle the lime contained in the cement. The very soft waters of the drainage basin which flowed in the shaft absorbed the lime.

The vibration of the concrete cast between sheet and rock eliminated the air contained in the concrete and increased the compactness of the concrete.

The data given in Fig. 17 could be completed as followed. The adit had been positioned at 1,040-m level (3,415 ft) to conform with the conditions of the ground and so as to divide the total length of the shaft into two nearly equal parts. Drainage of percolating waters was foreseen by creating a drainage room at the end of the adit. That room would have been crossed by the metal lining in the open air. For that purpose, the thickness of the lining had been increased to 20 mm (0·788 in.) in order to take the whole internal pressure, whereas in the concreted and metal-lined shaft, the rock was supposed to relieve the metal lining by absorbing about two-thirds of the internal pressure.

While opening the shaft, an inclined water-bearing fault had been found, which was not shown in Fig. 17. It cut across the axis of the shaft at the 1,077-m level (3,535 ft). It was then decided to extend the gallery 1,040 m (3,415 ft) with an inclined portion (shown in Fig. 17) which passed under the shaft. That portion ended in an injection room. The fault cut the axis of that inclined gallery at the 1,063·80-m level (3,490 ft), so that the fault plane was nearly perpendicular to the axis of the shaft and to the axis of the inclined gallery.

Encountering the fault had allowed the ground-waters to be drained in the inclined gallery and then in the adit. The lining which it had been intended to end at the 1,056-m level (3,465 ft) had been prolonged up to the 1,103-m level (3,620 ft), in order to join the section with reinforced lining to the place where the fault cut the axis of the shaft, and still to have a plated length of about 50 m (54·8 yd) above that point.

From the injection room at the end of the gallery, three fan-shaped borings were constructed at depths of 30, 26, and 15 m (32·8, 28·4, 16·4 yd). Those borings had shown that the rock was perfectly dry and compact above the fault and that it was not necessary to make any more injections in that area. Four injections, 20 m (21·85 yd) deep, had been made from the room at about 1,040-m level (3,415 ft) for the passage of the open-air pipe. Those had consumed only a small quantity of cement—300 kg (662 lb.) altogether. Those four injections had been made in order to avoid the infiltration of drainage waters to the lower part of the shaft. Therefore, when the injections had been executed, the room was entirely sheeted up to the adit level, 1,040 m (3,415 ft).

That created a natural drainage zone above the adit level (1,040 m), leading the drainage water into that adit. It also made two watertight zones surrounding the drainage zone, one being above (sound rock) and the other below (injected rock around the shaft). The main object was to reduce to a maximum known value the external pressure caused by the infiltration waters at the bottom of the sheeted shaft.

On p. 150 it was pointed out that the maximum internal water pressure was 18 kg/sq. cm (256·02 lb/sq. in.). That was correct for the lower part of the shaft without sheeting. At the bottom of the sheeted shaft the maximum internal static water pressure was 48 kg/sq. cm (682·71 lb/sq. in.). For calculating the sheeting required the latter value had been raised by 10% to make allowance for water-hammer effects caused by the gating of the turbines. Inclusion of the surge tank had given a maximum amplitude of 1,289 m (4,230 ft), which corresponded to a maximum internal dynamic pressure of 49 kg/sq. cm (696·94 lb/sq. in.) during the mass oscillations. For a complete discharge of both groups, a maximum instantaneous pressure of 52·5 kg/sq. cm (746·72 lb/sq. in.) was found from the water-hammer tests which corresponded to an increase of almost 10% in the static pressure.

The injections carried out in the different parts of the pressure shaft consumed, per linear metre, the quantities of material shown in Table 10.

TABLE 10

	Length of section :		Number of holes	Weight of grout per unit length	
	m	yd		kg/m	lb/ft
Section without sheeting (pozzolanic cement)	246·61	270	98	223·67	150·5
Sheeted section, inclined part	516·90	564	214	313·0	210
Slightly inclined part	85·60	93·7	29	1182·12	794
Turbine's collector	64·57	70·6	4	353·08	237

The rather high quantity absorbed by the slightly inclined section arose from the fact that the concreting had been carried out with a pump and that it was difficult to fill completely the interstices of the rock face in the upper part of the excavation. The injections had thus completed the filling done with the pump.

Mr F. L. Lawton, of Montreal, stated that the Brechbergmuehle plant built in Germany in 1907 and Porjus in Sweden in 1914 had commonly been accepted as the first underground hydro-electric stations in the world. However, the State of Washington, U.S.A., had its 270-ft-head Snoqualmie Falls plant completed in 1898, with a capacity of 10,000 h.p. There turbines, generators, and control boards were located in a chamber 200 ft long, 40 ft wide, and 30 ft high hewn out of solid basaltic rock. The plant was placed underground to avoid excessive spray and ice conditions arising from its proximity to the falls. It had been in successful operation ever since 1898, and experience had shown the plant to be clean, dry, and well-ventilated. The power-house chamber was unlined, the solid rock making that unnecessary.

The Author might have added that a factor influencing a decision to develop a particular power site as an underground station was frequently foundation conditions. That was particularly true where the plant was located in a valley filled with detritus or glacial-fluvial deposits.

As always with new developments, confusion appeared to be growing up in the language used, no doubt as a result of difficulties of translation from other languages into English. For instance, the Author used " pressure galleries " to describe tunnels operated under hydrostatic pressure. Would it not be simpler and more effective to use " pressure tunnel " ? Similarly, " steel-lined shaft " was used to describe what was nothing more or less than a pipeline embedded in a rock formation. It would seem "pressure conduit" would be more realistic. The term would be qualified by an adjectival expression covering the lining.

Referring to the rule-of-thumb method still applied in the design of steel-lined pressure conduits traversing heterogeneous and somewhat unreliable rock, the implication was that it was not practicable to apply appropriate remedial measures to such rock. However, grouting could normally be used to consolidate otherwise bad ground and produce, at a cost which could be estimated, satisfactory conditions.

At that point it might be desirable to draw attention to a few errors in certain of the Tables, with respect to the Kemano development.[94–97] In Table 3, the statement under "Anchorages" should read "Prepakt concrete, ½ to 3 in. placed by 10-in. pipe." A similar correction was necessary in the footnotes also. In Table 4, the ASTM A285 fire-box quality grade B steel used at Kemano had an ultimate strength of 3,650–4,560 kg/sq. cm and 23–29 tons/sq. in., whilst the ASTM A201 fire-box quality grade A had an ultimate strength of 3,490–4,680 kg/sq. cm and 22–30 tons/sq. in. In Table 5, discharge in cumecs was 4 × 62·5. In Table 6, the discharge per shaft was 68·0 cumecs and 2,400 cusecs, that value being the maximum discharge under operating conditions. In Table 7, the volume of excavation at Kemano for eight units, the service and control bay in the middle, and the major part of that for the ninth unit was 278,000 cu. yd, excluding 54,000 cu. yd for the access, ventilation, and cable tunnels. Excavation for the valve house, a separate chamber, was 21,000 cu. yd. In Table 8, the total height of the excavation, including discharge pit, should be 42·4 m. The total width of the excavation was 24·8 m. The total length for the first stage, including the service and control bay, was 195·5 m and for the complete development about 344 m. Crane span was 15·25 m. Distance between turbine centres was 18·3 m.

The Author noted that "Up to the present no failure of steel linings arising from the development of rock fissures has been reported. But failures of the lining have been caused by external water pressure, by grossly defective design, or by lack of homogeneity of the concrete filling between steel lining and the rock." It might be of interest to refer to work being done in connexion with the Yale project in the State of Washington, U.S.A. There the two 18-ft-6-in.-dia. tunnels were originally concrete lined except for a distance of about 250 ft near the power house where a steel casing was employed. However, hairline cracks, attributed to unpredictable movements, were found in the columnar basalt formation through which the tunnels had been driven. As a consequence, a steel liner was being installed inside the original concrete liner and the space between the two grouted. The tunnels were designed for a total pressure head of 330 ft.

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[94] References 94 *et seq.* are given at the end of the Author's reply on p. 596.

The Author quoted experience with the thin corrugated steel plates used for lining three pressure conduits designed by Marinoni for the Pallanzeno, Rovesca, and Mese power plants, as demonstrating that in some cases the whole hydrostatic load of a pressure conduit could be transmitted to the rock, the steel lining serving only as a watertight sheath. The Author further stated that " A tunnel or shaft will be safe if the resultant stresses at any point in the steel lining or at any point of the rock, including the effect of prestressing, are well inside the known permissible values." That was perfectly true and the Author recognized the weakness of the situation, from a design point of view, by his statement " Rock is by far the most important material and very little is yet known about its behaviour in situ." That was only too true, and warranted a strong plea that designers of underground power stations and pressure conduits should make known to the profession the bases of their designs and any particular difficulties encountered, since it was only by that interchange of knowledge that advances could be made.

In selecting the thickness of the steel liner for a pressure conduit one of the most difficult factors to contend with was the rock. Many features required consideration, among others the jointing, stratification, attitude, residual stresses, variation in strength of the rock, and in its modulus of elasticity. Even sound rock was disturbed in the belt surrounding the excavation by blasting operations, re-adjustment of stresses, and other factors.

In the case of the Kemano pressure conduit, the steel liner was given a thickness sufficient to carry the internal hydrostatic pressures as a free pipe at normal stresses, at the upper and lower ends, and the concrete backfill functioned only to protect the outside of the liner against corrosion and to provide a continuous support. Between the extreme ends of the pressure conduit, part of the hydrostatic pressure load was taken by the rock, thus keeping the steel stresses near acceptable working values for the particular steel employed. As a result, the concrete backfill between the liner and the rock not only protected and supported the liner but acted also as a compression member transferring part of the internal load to the rock. In order to meet that requirement, the concrete must be durable, of medium but uniform strength, and completely free of voids or segregation, which could lead to concentrations of stress and to excessive strain in the steel. The average thickness of the concrete backfill was about 3 ft. To secure the necessary tightness of contact between the liner and the concrete backfill, and between the concrete backfill and the rock surfaces, it was desirable to have the minimum of curing shrinkage in the concrete. For that and other reasons, the intrusion-Prepakt method was selected for the concrete backfill.

The aggregate used on the sloping sections of the pressure conduit was placed by means of a 10-in. pipe located on the top centre-line of the liner and filled with aggregate from hopper cars at the head of the shaft.

With that pipe maintained full of aggregate, gates were burned in the sides as filling of the annular space moved upwards. An average rate of aggregate placement of 3 ft per hour of pressure-conduit length was secured.

Backfill concrete was first placed on the sloping upper leg of the pressure conduit in the lower 400 ft. With that initial experience, the upper leg was then completed in one lift of 800 ft, whilst the lower leg was filled in a single lift of 1,940 ft. The mortar pumped into the voids of the coarse aggregate backfill consisted of approximately 7·15 parts by weight of sand (fineness modulus between 1·6 and 2·0), 5·0 parts type II (low-heat) Portland cement, 2·16 parts Alfesil, 0·07 parts of intrusion aid, and 3·15 parts of water.

Grouting was carried out with the aid of pumping equipment located at El. 210 for the lower leg and at El. 1,685 for the upper leg, with booster pumps as needed at intermediate points between the grouting skips and the bottom of the leg. Grout was injected into four holes at a time in such a manner that the grout surface advanced on a slight slope downwards towards the invert of the shaft. About 4,700 cu. yd of grout were required to fill the voids in the coarse aggregate. A total of 11,090 cu. yd of completed concrete were placed in the lower leg in 20 days, making an average progress of 4·0 ft/hour. Measured temperature rise in the concrete was from 48°F to 75°F and back to 50°F in about 14 days.

That the method used for emplacement of the concrete encasement was sound was demonstrated by about 1,000 cores taken after 3 weeks, most of which were located in the lower leg. Those cores showed an absence of voids and segregation, which could lead to concentrations of stress and excess strains in the steel liner. Moreover, very little grout was taken in final grouting operations when attempting to consolidate the rock or tighten up the concrete at pressures up to 300 lb/sq. in.

Many approaches could be made to the problem of pressure-conduit design. The rock mass surrounding the pressure conduit could be considered as a thick-walled cylinder enclosing a " cracked " zone immediately adjacent to the liner, with that cracked zone assumed to act as a series of columns between the exterior of the liner and the uncracked rock cylinder at a distance. Another method was predicated on application of Poisson's ratio to calculated compressive stresses in the rock along the extension of a horizontal diameter of the liner. In that case, the concrete sheath acted only as a strut between the active liner and the resisting rock, through which a part of the hydrostatic load was transferred outwards until balanced by internal stresses in the rock and the strain was reduced to zero.

Because of the variation in rock properties, almost the only practical way in which a measure of the modulus of elasticity could be secured was by tests on rock in situ. For that purpose, at Kemano, as soon as possible following the completion of the exploratory tunnel into the approximate region of the power house in 1951, a 10-ft-dia. hollow sphere of $\frac{1}{2}$-in.

steel plate was emplaced under conditions analagous to those to be used for the pressure-conduit liner proper. That sphere was provided with numerous gauges, of several types, in order to obtain a measure of the modulus of elasticity of the rock and the action of a thin-walled vessel encased in concrete and rock. The sphere was tested by pumping water into it, under pressures gradually increasing up to 3,600 lb/sq. in., 292% of the maximum dynamic pressure. That ½-in.-thick steel-plate sphere sustained the pressures entirely satisfactorily and justified the basic concept that a thin steel liner, backed up at all points by concrete and sound rock of sufficient thickness, would perform satisfactorily. Although some unsatisfactory results were obtained with the electrical-resistance gauges, the modulus of elasticity of the rock was ascertained to be about 2 million lb/sq. in.

The Kemano pressure conduits were emplaced in a granodiorite body forming part of the coast-range batholith. Although small feldspar, aplite, lamprophyre, and diorite dykes occurred the ground proved to be quite good.

Initial proof testing on the first pressure conduit at Kemano involved measurements of the stress in the liner in the 11-ft-dia. pressure-conduit sections at El. 210 and El. 1,685 by two sets of SR 4 strain gauges and mechanical extensometers devised to measure diametral changes. Excellent correlation between the results from the SR 4 gauges and the extensometers was secured at El. 210 but results at El. 1,685 were not as satisfactory as hoped because of some imperfections in the gauges. However, the results showed that the rock was taking at El. 210 68% of the stress and at El. 1,685 about 64%.

Mr Lawton agreed with the Author that " At the present time, most designers agree that the external water pressure is a major problem, but opinions diverge considerably regarding the distribution of the external pressure and even more so regarding the methods for stiffening the steel lining and the design of the lining against buckling." One of the difficulties in that connexion was that relatively little was known about the magnitudes of those external water pressures. It was, of course, assumed that no pressure-water escaped from the pressure conduit ; that was the basic purpose of the liner. Thus, the external water pressure which had to be taken into acccount in design, when the pressure conduit was empty, was that in the surrounding rock mass. In most cases, pressure conduits were located relatively close to the shoulder or slope of a major rock mass. Hence, because of the fissures and other joints which normally existed in such locations, ground-water pressures affecting pressure-conduit liners were likely to be relatively low.

That was one aspect of pressure-conduit design wherein publication of ground-water pressures found in the vicinity of pressure conduits, during investigation and during construction, would be of invaluable assistance to the profession.

It was predicted that future developments of underground hydro-electric stations would involve even larger diameter pressure conduits with thinner steel liners than now used, as the properties of rock in situ became better recognized, the interactions between liner and rock mass became better understood, and construction techniques were improved, with marked economies in necessary investment.

Mr A. J. Harris observed that the following details of a prestressed concrete pressure tunnel in North Africa might be of interest as an amplification of Dr Jaeger's description of precast concrete tunnel linings prestressed by grout injection. That tunnel was designed by Mr Freyssinet.

The tunnel was of 8-ft-6-in. internal diameter and supplied about 7 cu. m/sec of water at a pressure of 43 lb/sq. in. The tunnel was about 160 ft deep and the subsoil was a sandy clay of varying consistency.

The precast concrete lining was composed of rings, 20 in. long and 2¾ in. thick, each ring being divided into three segments. Those segments were made in steel moulds, the concrete being of high quality and the finish to a high standard. Each segment weighed approximately 530 lb.

The principle of the lining was as followed. The segments were placed in position by means of a special machine; they rested on carefully levelled blocks at the base and were wedged against the wall of the tunnel by loose rock packed in. On completion of a convenient length, a temporary longitudinal prestress of about 60 tons was applied by tie-bars lying on the inside. The end was then closed by an airbag which was inflated between the end ring and a prepared surface on the face of the rock. The space between the rings and the rock face (1 in. to 1½ in.) was injected with colloidal grout, a pressure of 15 lb/sq. in. being applied at first to fill the space and the pressure then being raised to 65 lb/sq. in. to prestress the rings. The joints were everywhere left open, the grout lost water copiously and attained a high degree of density, and its set was accelerated. The pressure was maintained for about 20 min to permit full dewatering and to ensure that the commencement of the set occurred under compression.

Where the rock was unstable and required supporting before the placing of the lining, similar rings of larger diameter were fitted and packed against the rock face; the final lining passed through those supports and the operations of injection and completion were exactly the same. Those supporting rings were closely spaced when the rock was extremely bad; elsewhere they were at open centres. If the tunnel surface was irregular, the irregularities were packed with coarse aggregate as each ring was placed; full penetration of that aggregate by the colloidal grout was obtained.

On completion of following lengths, the temporary prestress was released, the concrete thus tending to expand longitudinally and thrust against the lengths on either side. There was thus an appreciable residual longitudinal prestress.

Extensive strain measurements, both during and after injection, were made on the inner surface of the lining; contractions of the order of 3 to 3·5 mm were usual followed by a slight relaxation of 0·4-mm maximum. The Young's modulus deduced from measurements during injection varied between 5×10^6 and 6×10^6 lb/sq. in.; the curves from which those values were deduced showed remarkable consistency.

The pressure was occasionally raised to 85 lb/sq. in. when the concrete stress was 1,700 lb/sq. in.; even with that stress the curve was still elastic and showed no signs of flattening out. On certain test lengths the pressure was then released; apart from a slight displacement arising from the time lag in decompression there was satisfactory agreement between the rising curve and the falling curve.

Two lengths of tunnel were built using that technique, the one described above of 8·5 km at Oued Fodda and a previous length of 5 km (10-ft dia.) at Kerata, in which the methods were slightly different.

The technique had also been used for vertical shafts, notably for the surge tanks and access shafts at Oued Fodda, whose total height was 600 ft.

Mr G. T. Colebatch, of Hobart, Tasmania, said the Paper covered a wide range of problems of interest to the Hydro-Electric Commission of Tasmania. Of particular interest was the section on the buckling of steel linings by outside water pressure and it was on that subject that the following statements were made.

The Trevallyn Power Development, which was approaching completion near Launceston in Northern Tasmania, involved both high and low pressure steel tunnel linings. The steel-lined tunnels were sensibly horizontal and were in dolerite of varying quality; some sections were very sound whilst others had required considerable support. Good and bad sections alternated and were independent of the depth of cover. Experience showed that local weathering might extend to the surface 200 ft above. Furthermore, during the excavation of the tunnels, a number of ground-water sources were tapped yielding thousands of cu. ft of water and indicating extensive underground storages. Accordingly, high external water pressures were considered possible on both the high- and the low-pressure lining.

The low-pressure lining, 14 ft 3 in. in diameter and of $\frac{1}{2}$-in. plate, was stressed as a free pipe to a maximum of 16,000 lb/sq. in. Twenty-foot lengths were concreted at 24-hour intervals, using brick bulkheads, and the lining was protected against ground-water pressure by a drainage system and a system of hooked bars which fastened it to the concrete.

The drainage system consisted of two 6-in.-dia. asbestos-cement pipes, one set each side of the lining on the tunnel invert. On the completion of grouting, holes at 10-ft centres were drilled from within the lining to pierce the drains and extend about 8 ft into the rock beyond. That part of each hole, between the steel and the 6-in. pipe, was plugged with mortar and the hole in the lining sealed. The drains extended outside the tunnel where they discharged to waste. Since they were provided with rubber joint-rings, the drains were considered proof against the intrusion of grout but, as an additional precaution, they were flushed during concreting and grouting.

The hooked bars, $\frac{9}{16}$ in. in diameter and 9 in. overall, were set at 6-in. centres in sixteen equally spaced rows parallel to the lining axis. The bars were stud-welded normal to the lining, a man fastening a row of forty bars in 8 min.

The lining was assumed restrained along the rows of hooked bars and so divided into sixteen thin arches with restrained ends, each subjected to uniform external pressure. The relevant equations derived, assuming no buckling, were :

$$\frac{f_a}{q} = \frac{\delta}{12R} \cdot \frac{\alpha}{\beta - \gamma \frac{\delta^2}{12R^2}} \pm \frac{\epsilon}{2\left(\beta - \gamma \frac{\delta^2}{12R^2}\right)} + \frac{R}{\delta}$$

$$\frac{V_a}{q} = \frac{\delta^2}{12R} \cdot \frac{\alpha \tan \alpha}{\beta - \gamma \frac{\delta^2}{12R^2}}$$

Where f_a denoted the maximum stress, which occurred at the support, in lb. per square inch.

V_a denoted the radial shear at the support in lb/unit length.

$$\beta = \tan \alpha - \frac{\alpha^2}{\sin 2\alpha} - \frac{\alpha}{2}$$

$$\gamma = \frac{\alpha^2}{\sin 2\alpha} + \frac{\alpha}{2}$$

$$\epsilon = \alpha - \tan \alpha$$

R, α, δ, and q were as indicated in Fig. 47.

Additional stresses were caused by any small gap between steel and concrete as a result of temperature effects.

Those were given by :

$$\frac{f_a}{q'} = \frac{\delta}{12R} \cdot \frac{\alpha}{\beta - \gamma \frac{\delta^2}{12R^2}} \pm \frac{\epsilon}{2\left(\beta - \gamma \frac{\delta^2}{12R^2}\right)}$$

$$\frac{V_a}{q'} = \frac{\delta^2}{12R} \cdot \frac{\alpha \tan \alpha}{\beta - \gamma \frac{\delta^2}{12R^2}}$$

where $$K_o = \frac{q'R^2}{\delta E^*}$$

i.e., q' was the uniform external pressure which, in the absence of the hooks, would cause a uniform gap K_o.

Equation 130 of "The Theory of Elastic Stability," by Timoshenko was used to assess the safety of the arches against buckling, thus :

$$\frac{\sigma_{cr}}{E^*} = \frac{k^2 - 1}{12}\left(\frac{\delta}{R}\right)^2$$

where

$$k \tan \frac{\pi}{n} \cot k \frac{\pi}{n} = 1$$

and σ_{cr} denoted the buckling stress in lb/sq. in.

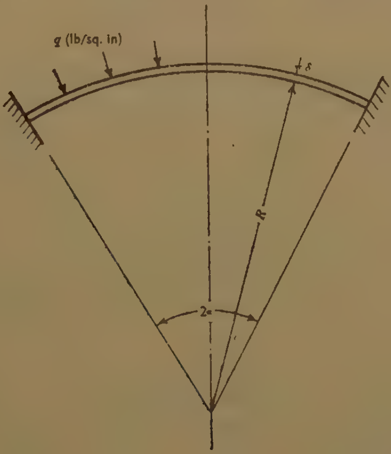

FIG. 47

That equation assumed a truly circular arch before buckling and so must be applied with a large factor of safety. Furthermore, the value of σ_{cr} must be less than the yield stress to be significant. The permissible mean compressive stress was taken as no greater than one quarter of the buckling stress.

For any external pressure the stress f_a first increased and then decreased as the number of rows of anchorages was increased. Also, as the number of rows of anchorages increased, the mean stress decreased, significantly so when the number was high, and the radial load carried by the hooked bars increased. So, for high external pressures, a large number of rows of anchorages were required and the load on those anchorages was high but the stress in the shell was low, except at its junction with the anchorages, and the safety against buckling was great.

Because of temperature effects the value of K_o assumed at Trevallyn was 0·01 in. A maximum external head of 100 ft was assumed and

$\frac{1}{16}$ in. was deducted from the lining thickness to allow for possible corrosion. The resulting stresses were shown in Table 11.

TABLE 11

	Stress : lb/sq. in.	K_0 : in.	δ : in.
f_a	20,000	0·01	0·437
σ'_{cr}	33,000	0·01	0·437
Mean compressive stress σ .	8,000	—	0·437
Radial anchor stress . .	5,500	—	0·500

The diameter and spacing of the hooks in the rows was not governed by radial loads but by the desire to reduce stresses at the junction of the anchorages with the shell and by doubts as to the shear occurring there. It was to be noted that, if there was a non-uniform yielding of the rock and the concrete with it, high circumferential shear could occur at the junctions of the anchorages with the lining. For that reason that type of stiffening, which was far cheaper in Tasmania than equivalent stiffening by steel girders, appeared desirable only when the steel stress under internal pressure was low or where the rock appeared of uniform character in all directions.

Concrete was placed by pump delivering to the crown only and little trouble was experienced except at the invert where it was found impossible to avoid some drumminess. Those drummy patches occurred despite the efforts of men working with vibrators in the confined space at the invert behind the lining. It was hard to see how they could be avoided since vibration brought a water film to the steel surface which was trapped at the invert and settlement shrinkage subsequently increased the gap so formed. The gaps were very shallow but probably considerably greater than any gaps caused by temperature effects.

The high-pressure lining, 13 ft 6 in. in diameter and of ⅝-in. plate, was stressed as a free pipe beyond the yield point. Uniform yielding of the rock under internal pressure could not be assumed and therefore the anchorage system of the low-pressure lining was not favoured. Protection was provided by two drainage systems without any stiffening or anchoring of the lining to the concrete.

The first drainage system was identical to that provided for the low-pressure lining, i.e., 6-in.-dia. asbestos-cement pipes pierced by drill-holes on completion of grouting.

The second system involved a series of 1½-in.-dia. steel pipes at 10-ft centres, encircling the lining with a clearance of 1 in. and each connected to a 4-in.-dia. steel pipe leading to the power house. After grouting, each 1½-in. pipe was pierced by twelve equally spaced holes made by a ½-in.-dia.

drill inserted through holes previously drilled in the steel lining. The holes in the lining were then sealed. The system was considered proof against the high grout pressures applied but, as an additional safeguard, the pipes were flushed during concreting and grouting.

Ground-water forcing the shell away from the concrete would create a shallow indentation which would spread until it reached a hole leading to a circumferential pipe whence it would pass into the 4-in.-dia. pipe and so to waste. Conservatively estimated inflows could pass without overstressing the lining.

The 4-in. steel pipe and the asbestos-cement pipes passed into the power house where valves and gauges enabled pressures and flows to be observed. The outlet from the 4-in. pipe would be sealed except when the tunnel was dewatered. The 6-in. pipes would always drain freely since their purpose was to limit the flow to be dealt with by the second system on dewatering the tunnel.

The drainage system extended from downstream of a short length of lining heavily stiffened by ring girders to within 500 ft of the power house, where the lining was designed as a free pipe and considered capable of withstanding the external pressure anticipated.

Mr A. Hutter and Mr J. Hanimann, of Zurich, Switzerland, gave details of the project at Fionnay, which presented some unusual problems in the design and construction of pressure galleries, steel-lined pressure shafts, and underground power stations.

The reservoir had a maximum level of 6,400 ft above mean sea level, and the net volume between levels 6,400 and 5,900 ft was 232 million cu. yd. The arch dam was 770 ft high, crest length 1,760 ft, and the volume of concrete used in its construction was 2¾ million cu. yd. The metric lengths of the shafts were shown in Fig. 48a. Other details were as followed:—

Pressure gallery: diameter 10·5 ft
length 15,650 ft
maximum pressure at the inlet 238 lb/sq. in.
at the end 290 lb/sq. in.
Steel-lined pressure shaft: diameter 7·9 ft
length 1,450 ft
gradient 0·8 in 1
Surge-tank shafts: diameter 13·15 ft
length 985 ft
gradient 0·8 in 1
steel-lined for 430 ft

The underground power station had two (ultimately to be increased to three) vertical-shaft units with Francis turbines, 750 r.p.m., each unit rated at 42,500 kW and discharging 405 cusecs at a maximum gross head of 1,555 ft (at present the highest head permitted for Francis turbines).

The lower stage at Riddes was served from a compensation basin at Fionnay having a net volume of 236,000 cu. yd. between levels 4,870 and 4,910 ft above mean sea-level. Other details were :

Pressure gallery :	diameter 10·2–10·7 ft length 48,300 ft maximum pressure 125 lb/sq.in.
Two open penstocks :	diameter 4·95–5·6 ft length 5,820 ft

Riddes power-house had five horizontal-shaft units each with two Pelton wheels running at 500 r.p.m., each unit rated at 45,000 kW and consuming 202 cusecs. The maximum gross head was 3,340 ft. The tail-race culvert was 4,000 ft long, temporarily working under pressure.

The whole development was actually under construction. The upper stage would probably go into operation towards the end of 1955, the lower stage about 2 years later. A short description had been published.[98]

The hydraulic system of Fionnay stage corresponded to type 3, described by the Author on p. 119, with Francis turbines, Fig. 48b.

The pressure gallery at Fionnay was subjected to a maximum internal pressure of 290 lb/sq. in. The main part of it was in very sound rock (" Casanna schists ").

The alignment was traced in order to obtain a high overburden, thus involving a length of the intermediate adit of 2,300 ft.

To investigate the behaviour of the rock under high pressure, two test galleries in Casanna schist were driven with a diameter of 10·5 ft (as in the main gallery), each 100 feet long.

The first one was normally lined with concrete 10 in. thick and was grouted under a pressure of 426 lb/sq. in. After some rapid variations of pressure caused by surge-tank oscillations, the pressure was maintained at 284 lb/sq. in. At that pressure, the loss of water amounted to 0·0246 gal/min/sq. yd.

After application of a 3-in. layer of reinforced gunite (sectional area of reinforcing bars : 3·5 sq. in./yd), the loss was reduced to one-fourth (0·0062 gal/min/sq. yd).

The second test gallery was provided with a free space outside the ordinary 10-in. concrete lining by use of a net of expanded metal (as described by the Author on p. 150). That space was grouted with a special mortar under a pressure of 398 lb/sq. in. The difficulties encountered with that method had led to the decision that the main gallery was to be constructed by the usual method with a 10-in. concrete lining (Portland-cement content 600 lb/cu. yd). The grouting was done at a pressure of 568 lb/sq. in., i.e., twice the maximum internal working pressure.

Concreting and grouting had already been completed. During the

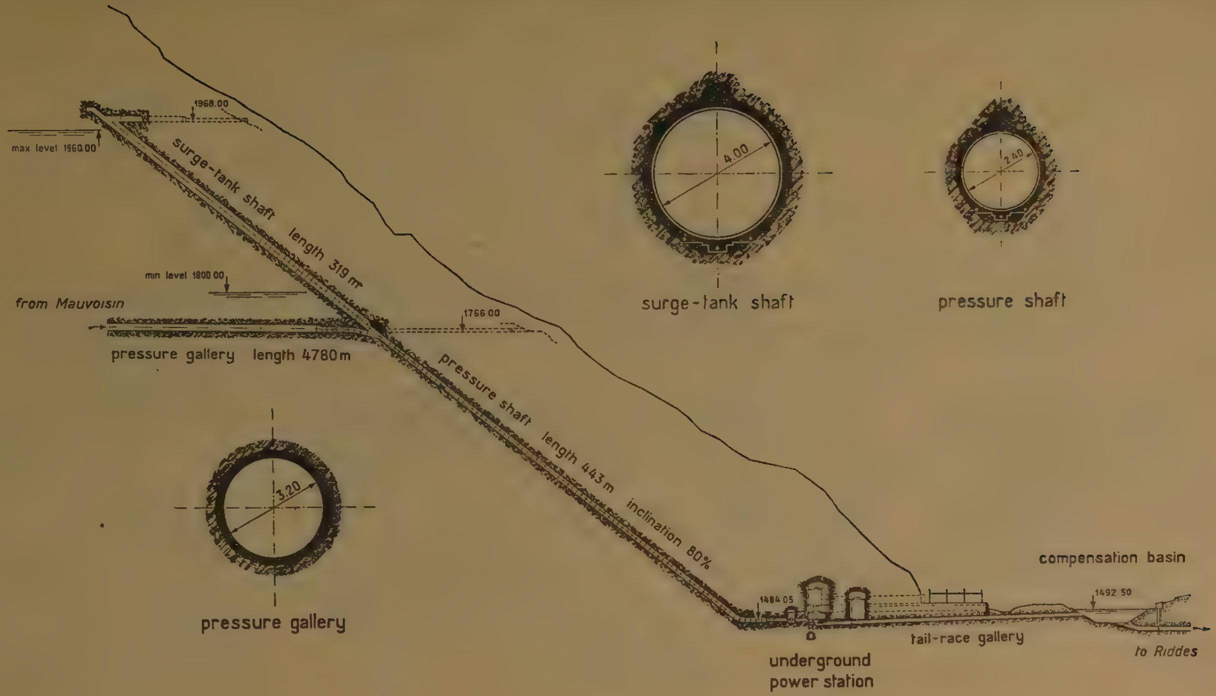

FIG. 48a.—MAUVOISIN SCHEME (FIONNAY STAGE). GENERAL ARRANGEMENT

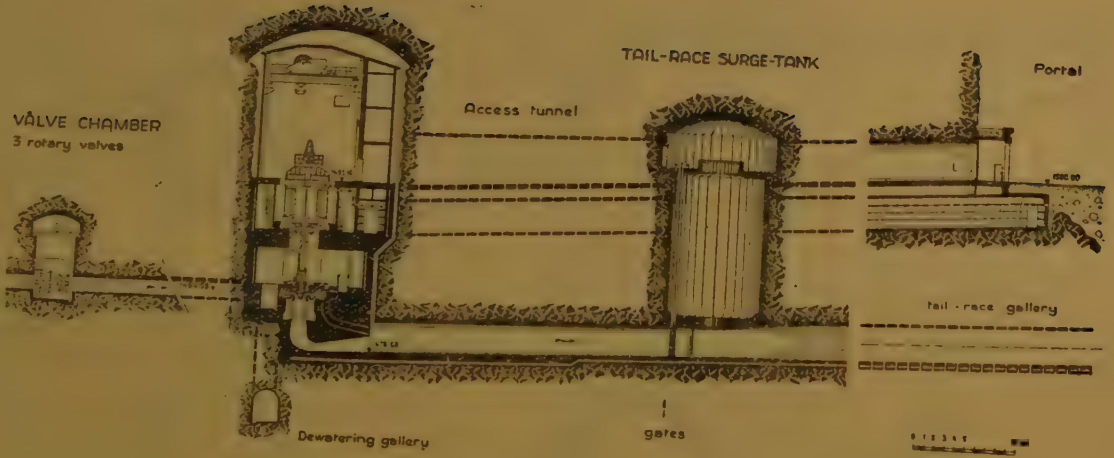

Fig 48b.—Mauvoisin. Longitudinal section of underground power station. (See Fig. 43a)

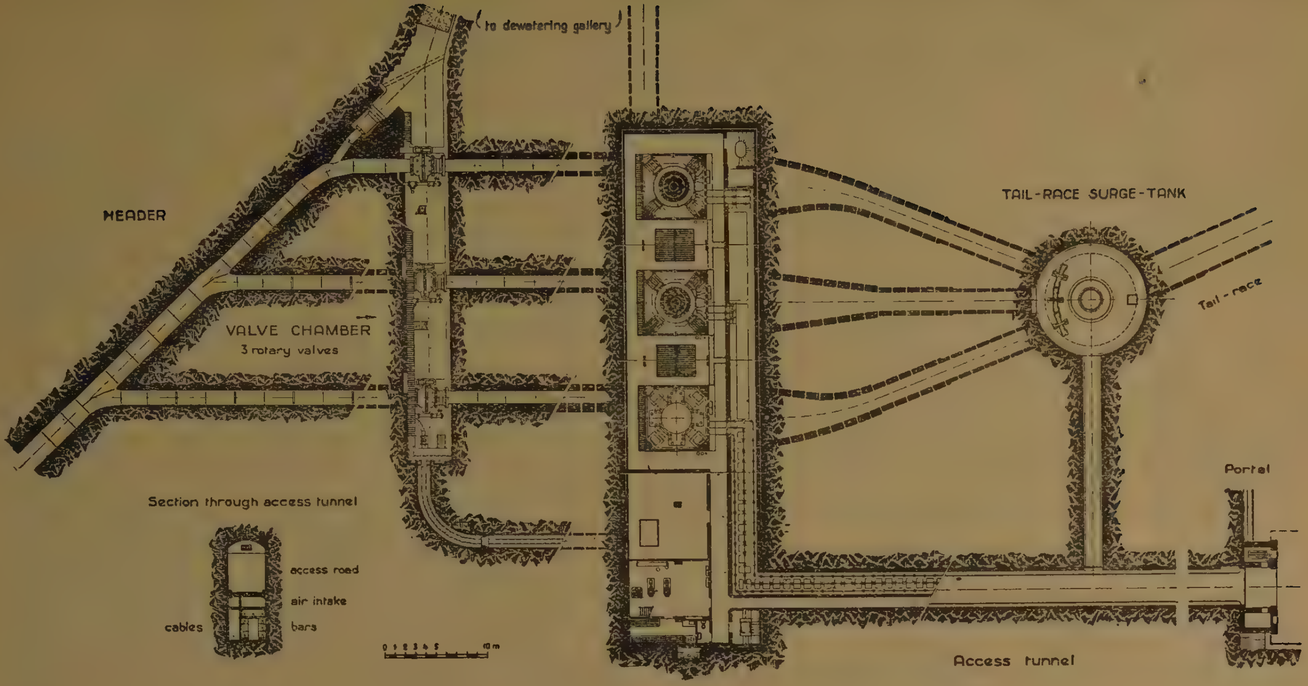

FIG. 48c.—MAUVOISIN. PLAN OF UNDERGROUND POWER STATION (SEE FIG. 43a)

next few months, the gallery would be submitted to systematic pressure tests to determine accurately the losses of water and the deformations in the rock. The gallery would be divided into two test sections according to the prevailing rock qualities.

It was intended to repeat the grouting to fill up possible cracks in the concrete lining and between concrete and rock (caused by plastic rock deformation). The decision to line sections with reinforced gunite would depend upon the loss of water.

The closing devices, usually located at the lower ends of pressure galleries, were dispensed with at Fionnay. So far as was known, that feature was successfully adopted for the first time at the Swiss hydro-electric power development of Calancasca.[99]

Closing devices at the lower ends of pressure galleries were located in inaccessible places and it was frequently very difficult to locate and correct any failures or other troubles, especially in winter time. Their elimination therefore was an important operational advantage. However, that disposition was only permissible under two conditions : presence of dependable automatic closing devices at the inlet of the pressure tunnel ; and provision of ample draining facilities in the valve chamber near the power station.

The elimination of closing devices at the lower end of pressure galleries provided more freedom in the disposition of the surge tank. At Fionnay, it was a partially steel-lined shaft of 13·3 ft diameter in direct alignment with the pressure shaft and an upper expansion chamber. That layout protected the pressure gallery effectively from water-hammer and permitted the use of the same inspection device (winch, rope, and carriage) for both shafts.

Compared with a vertical shaft, the inclination allowed for a reduction in diameter with the same hydraulic effect. The same strength against buckling of the steel lining was obtained with less wall thickness.

The same layout of a surge shaft, however, with a lower expansion chamber placed directly above the pressure gallery, was also adopted at the above-mentioned Calancasca scheme.

At Fionnay, it was possible to dispense with the lower chamber because at the lowest range of reservoir level, its net volume was greatly reduced ; and so, after a sudden output surge, a successive loading of the groups was permissible.

The pressure shaft, surge tank, and header were also located in sound Casanna schist with little seepage water. The internal static pressure at the upper end of the pressure shaft was 284 lb/sq. in. and that at its lower end 677 lb/sq. in. (810 lb/sq. in. dynamic). The steel lining had an inner diameter of 7·9 ft. It was proof against buckling under an external hydrostatic pressure of 335 lb/sq. in. That value was determined after consideration of the overburden and the strike and dip of the rock beds. The buckling calculations resulted, in the lower part, in a plate thickness of 20 mm and a distance of 9·2 ft between stiffening rings (angle iron, section 120-by-120-by-13 mm); in the upper part in a plate thickness of 17 mm and a distance of 6·1 ft between the same rings. Those rings were welded in the shape of a roof on the outer surface of the lining. The space was thoroughly filled with mortar. The steel was of Siemens-Martin boiler-plate quality; its ultimate strength ranging from 22 to 28 tons/sq. in.

The lining of the surge-tank shaft (13·3 ft dia.) was calculated for an external pressure of 284 lb/sq. in. It involved a plate thickness of 20 mm and a distance between rings of 6·1 ft (angle iron, section 150-by-150-by-14 mm).

Owing to the fact that those rings were very expensive, there was a tendency to avoid their use for diameters less than 10 ft, but to employ an increased plate thickness. That had three further advantages: less excavation, less concrete volume, and improved concreting conditions.

The header was entirely embedded in concrete. Taking account of the proximity of the valve chamber and the power-station excavation, its plate thickness was calculated as for a free pipeline submitted to internal pressure. That thickness resisted buckling under 355 lb/sq. in. external pressure without stiffening rings. The lining consisted of chromium-copper steel (Dillinox 50) with an ultimate strength ranging from 32 to 39 tons/sq. in. All workshop and field welds were ground down flush with the plate surfaces and stress-relieved. The field welds were spot-checked with gamma rays using the isotopes of iridium 192.

In order to maintain the initial smoothness of the inner plate surface for a long time, an effective anti-corrosive treatment was adopted (sand-blasting, zinc spraying of 1,000 g/sq. m effective deposit, equivalent to a layer thickness of 0·2 mm, with local negative tolerance of 0·04 mm, two cold bitumen coats and a final hot bituminous coat 2 mm thick).

The operational safety of a steel-lined pressure shaft depended essentially on the quality of the concrete lining and grouting. To ensure those conditions, the following measures were taken:—

(1) Short steel liners were employed, each 18·5 ft long, separately welded and concrete placed around them.
(2) A rich mix was used for the concrete (500 lb. of Portland cement per cu. yd of hardened concrete).
(3) Mixed concrete was transported to the place of concreting in a container.
(4) The concrete was thoroughly vibrated. An additional excavation was provided along the top of the shaft to enable the man handling the vibrator to reach the lowest points.
(5) The above-mentioned stiffening rings were so designed as to facilitate embedment.
(6) In order to avoid cavities, all anchors, hedgehog hairs, and

hoop-irons vertical to the surface were abandoned from the start of the work.

(7) Two 2-in.-dia. steel pipelines were installed to drain water from the concreting area. Those draining pipes were to be filled with mortar during the grouting.

(8) The rock was grouted to a depth of about 8 ft (equal to the diameter of the shaft) and the contact surfaces between rock/concrete and concrete/steel lining with a pressure of 284 lb/sq. in. (i.e., 80% of the external hydrostatic pressure adopted for the buckling calculation). Two or three of those groutings had now been finished without the occurrence of any buckling.

The underground power station was placed in sound, almost vertically dipping Casanna schists. To ascertain the nature of the rock, some test galleries were driven at different levels parallel to the longitudinal axis of the future cavern. That axis was placed perpendicular to the strike of the rock beds. Fig. 48c.

The following basic design principles were adopted :—

(1) The layout was as straightforward as possible, with the minimum number of galleries.

(2) The valve chamber was separated from the power station (distance between centres 80 ft) for safety in case of rupture of a rotary valve.

(3) The transformers were outdoor (because of fire risk).

(4) The volume of excavation for the main cavity was kept to the minimum in order to reduce the possibility of rock deformations.

(5) The concrete vault was placed as soon as the space for it had been excavated; the vault protected all the later works.

(6) The generator room was constructed in structural steel in order to shorten the building time and to reduce the volume to be excavated.

(7) The walls and roof of the generator room and the annex above the generator floor were completely independent of the rock in order to avoid any influence of rock deformation. The heavily reinforced concrete slabs below the generator floor (up to 5·75 ft thick) were liable to damage in the event of extensive rock strains occurring.

(8) Good dewatering facilities for both valve chamber and power station were provided. The gallery connecting those two caverns was provided with a pressure-proof steel door.

In view of the good rock quality, the side walls of the power station above the generator-room floor were only gunited, except the wall with the hanging outcrops of rock beds, which was lined with an anchored reinforced concrete wall.

The gross excavation for the power station amounted to about 27,000 cu. ft, i.e., 0·21 cu. ft/kW.

Mr Hutter and Mr Hanimann then gave a list of information which could be added to the Author's Tables 1, 3, 4, 5, 7, and 8 as followed.

ADDITION TO TABLE 1

Power station	Country	Gross static head:		Internal pipe diameter:		Steel thickness:		Stresses in steel plate:	
		m	ft	m	ft	mm	in.	kg/sq.cm	lb/sq.in.
Calancasca	Switzerland	404	1,330	1·49	4·9	13	0·51	2,315	32,800
Fionnay	,,	471	1,555	2·40	7·9	20	0·79	2,830	40,200

ADDITION TO TABLE 3

Pressure shaft	Thickness of concrete:		Length of pipes:		Anchorages
	m	in.	m	ft	
Calancasca	0·20	8	6·40	21	None
Fionnay	0·25	10	5·60	18·4	None

ADDITION TO TABLE 4

	Ultimate strength:	
	kg/sq. cm	tons/sq. in.
Calancasca: Siemens-Martin boiler-plate quality I	3,500–4,400	22–28
Fionnay (header): Cr-Cu-steel quality Dillinox 50	5,000–6,000	32–39

ADDITION TO TABLE 5

Name	Country	Date of commission	Gross head: m	Internal dia: m	Rock	No. of pipe	Discharge: cumecs	Gradient: tan α
Calancasca	Switzerland	1949	404	1·49	Gneiss	1	6	0·80

ADDITION TO TABLE 7

Date of commission	Power station	Country	Capacity : kW	Gross head : ft	Turbines : *v.* = vertical	Volume : cu. yd	Excav. for valve house : cu. yd	Vol. of valve hous per kW installed : cu. yd
Constr.	Fionnay	Switzerland	127,500	1,555	3 Francis, *v.*	27,000	1,800 (*a*)	0·21(*a*)

ADDITION TO TABLE 8

Power house	Head : m	Total height: m	Total width : m	Total length: m	Crane span : m	Crane capacity : tons	Turbine horsepower	Distance between turbine centres : m	Position of transformers	Valves
Fionnay	471	24·30	12·68	49·80	8·20	80	3 × 72,000	11·20	Outdoor	Special excavation

Mr Samuel Judd, of Denver, Colorado, said the increased use of underground power plants had been due partly to improvements and developments in the technique of tunnelling, and partly to improvements in the design and construction of steel-lined shafts. Since those were two of the major items of expense in an underground-power-plant scheme, economies obtainable in tunnelling, shaft design, and construction affected the feasibility of the project as a whole.

The two principal arguments for an underground power plant were adverse climate and adverse topography. A suitable rock formation was an obvious requirement and was usually difficult to find. However, if a good open accessible site was available, a surface plant would be more economical than one placed in the rock. In a cold climate the construction season for the underground plant was year-round, thus expediting power delivery. Neither heating of the underground plant nor protection from the weather was required. That might have advantages over a surface plant. When surface scenic features must be preserved or where protection was required for military purposes the underground plant had a definite advantage.

Despite the trend towards underground power plants in many countries, the number of such plants that had been built in the United States, either by private or Government agencies, was very small. That was usually attributed to the relatively high labour costs in that country, since labour was of prime importance in the construction of that type of plant. The principal reason why underground power plants were not being built by the Bureau of Reclamation was because at the great majority of construction sites the advantages of climate and topography for an underground plant were not available. Sites with easy access and relatively inexpensive excavation gave an advantage to the surface plant. Cases here occurred where refrigeration requirements to counteract high underground temperatures, especially in warm climates, increased costs of the underground plant. The more expensive underground exploration increased costs. Elimination of the superstructure for the surface plant meant a saving in costs which had no parallel in the underground plant. The Bureau of Reclamation had been able to overcome partially the short construction season due to winter weather at some sites by utilizing superstructure frames of structural steel which could be erected and then covered in a temporary manner, or covered permanently by the use of prefabricated insulated sandwich wall and roof panels. That provided winter protection and made the superstructure crane available for erection work much sooner than if concrete frames and walls were used.

In recent years it had been the policy of the Bureau of Reclamation to include a cost study for an underground power plant in all feasibility studies where suitable rock formations were available. To date that organization had been able to construct a more economical power plant on the surface in every case although such an alternative scheme was still under consideration for the proposed Hells Canyon Project (Idaho) and the Trinity River Project (California). Since most of the sites were on major rivers and reasonably accessible and had a fairly mild climate, they offered greater advantages for surface construction. The site for the Eklutna Power plant (Alaska) was the most favourable of all for the underground scheme, but a surface plant had been constructed on a fairly open and accessible site after exploratory tunnels showed the rock to be sufficiently fractured to require extensive support.

A recent cost study which involved a detailed comparison of a surface plant as opposed to an underground plant was the feasibility study for a proposed third power plant at the Grand Coulee Dam in the State of Washington. The proposed plant would supply additional peaking capacity as well as additional capacity during periods of high run-off, when the spillway of the dam would otherwise be used. The power house in both schemes was to contain eight Francis turbines of 165,000 h.p., each at a rated head of 300 ft and with a discharge of 5,080 cusecs per unit.

Studies indicated that the best location for either scheme would be immediately downstream of the right abutment of the dam with the intake structure a short distance upstream of the same abutment. Two main power tunnels would by-pass the immediate abutment area. For the underground scheme the power tunnels would branch off to the eight turbines and the discharged water would be carried to the tail-race adjacent to the right power house by two tunnels. For the surface plant the power tunnels would terminate at portals with steel penstocks continuing down the rock slope and into the power plant. That arrangement would require considerable excavation for the surface plant. However, because of the low cost of removing the overburden, it proved to be the most economical arrangement. The overburden was a nespelem silt which was very unstable when saturated and would be excavated to a slope of 6 : 1 below normal high water and to a slope of 3 : 1 above that level. Because such a large excavation would eliminate the existing right switch-yard, the estimate included the cost of an entirely new switch-yard in another location.

Cofferdam costs were comparable for both schemes since both would involve alterations to the tail-race for the existing right power house to satisfy hydraulic requirements.

The costs of waterways alone for the two schemes were virtually identical despite the great physical differences between them. The costs of transmission line, switch-yards, etc., were lower for the underground scheme, mainly because of less interference with existing structures.

The underground plant would be located in the granitic bed-rock abutment with transformers located on the surface above. The length of tunnels for the low-tension leads would be about 480 ft, which was well within the distance of 250 to 300 m mentioned by the Author. The underground plant was moved as close to the channel as possible, since that arrangement provided the shortest length of waterway. The resulting overburden depth would be slightly less than would normally be considered desirable (average excavation would be 90 ft wide and shortest distance from roof excavation to surface above would be about 150 ft, measured at an angle of about 30° from the vertical). Because of the relatively shallow depth of overburden, it was felt that the " hydrostatic " rock condition described by the Author would not be present. However, because of the fractured condition of the rock it was felt necessary to use a system of grouted anchor-bars, drilled into the rock to insure stability, in a manner similar to that for rock bolts. The roof would be supported by a reinforced concrete arch, placed before the lower part of the excavation was started. For ventilating purposes a suspended ceiling would be constructed below the arch and the side walls faced with concrete.

The surface-plant location was fixed at the most economical point considering cost of tail-race against cost of steel penstocks.

The cost of the surface power-plant structure alone, not including transmission lines, switch-yards, etc., was found to be approximately one-half that for the underground structure. The total estimated cost, including transmission lines, switch-yards, etc., indicated that the surface scheme would be about 10% more economical.

Mr H. K. Brickey, of Denver, Colorado, observed that Dr Jaeger's Paper would be invaluable to engineers confronted with the problems of design of underground pressure conduits, shafts, and power plants.

In the past 10 years the Bureau of Reclamation had designed and constructed three pressure tunnels of 8 ft, 10 ft 6 in., and 12 ft 6 in. dia. for surface hydro-electric power plants on the Colorado–Big Thompson Project in the State of Colorado, and one 9-ft-dia. pressure tunnel leading to a surface hydro-electric plant on the Eklutna Project in Alaska. Also, the Bureau had recently designed the 21-ft-dia. Eucumbene-Tumut and T–1 pressure tunnels now under construction for power development by the Snowy Mountains Hydro-Electric Authority in Australia. Surge tanks and gate shafts had been integral parts of all those designs. Maximum heads on those tunnels ranged from 80 to 370 ft and maximum overburden ranged from 200 to 4,600 ft. The ratio of maximum overburden to maximum head for those tunnels ranged from about 1·5 to 26. All those tunnels were lined with concrete but did not have any steel liner to serve as a watertight membrane. In only one of those tunnels (the Prospect Mountain Pressure Tunnel in Colorado) had rock stresses been determined by strain-gauging methods for the purpose of elimination of steel-bar reinforcement in the concrete lining, as mentioned in reference 46. In the Eklutna Tunnel in Alaska where the maximum head was 175 ft and where overburden existed to approximately 4,600 ft above the tunnel, bar reinforcement was eliminated from certain reaches of the concrete tunnel lining by visual inspection of the rock after excavation had been completed.

Generally, pressure tunnels must be designed on the basis of interpretations from surface inspection and from local knowledge of geological formations for guides as to what materials and ground-water might be expected. Consequently, without knowledge of the forces produced by the geological formation of the earth's crust, and because of the variable elasticity of the materials involved, what was considered to be a conservative approach had been adopted in the original tunnel designs and those had been modified as deemed necessary for the local conditions encountered during construction.

Although the Bureau agreed with the practice of pressure grouting through the concrete lining to provide contact between the rock and concrete so that internal pressures might in part be resisted by the rock, pressure grouting had been omitted in reaches where ground-water under considerable head was encountered and which could build up a high external hydrostatic pressure on the concrete lining. In such reaches on the Eklutna Tunnel, for example, permanent drain pipes had been installed on both sides of the tunnel-invert excavation with risers at various intervals embedded in the concrete and terminated on the interior of the tunnel lining with flap valves, it being assumed that the external hydrostatic pressure would thus be equalized with the internal pressure. Objection to thin-steel watertight membrane lining in pressure tunnels was based on the belief in the danger of buckling which could occur whenever the external hydrostatic pressures might be built up on that membrane after completion of the tunnel. Anchorage of the steel lining in the concrete as proposed would alleviate that danger to some extent, but the steel liner was not preferred unless minor leakage through the concrete tunnel lining and overburden could create hazards or objectionable conditions.

The thickness of concrete tunnel lining had generally been selected on the basis of approximately 1 in. per foot of tunnel diameter to the excavation "pay line," with a minimum thickness of ¼ in. per ft of tunnel diameter, or sufficient thickness to give adequate cover for the bar reinforcement to be installed. Assuming some rock resistance after pressure grouting, even in poor quality rock, the amount of bar reinforcement required to withstand the maximum internal hydrostatic pressure was determined on the basis of steel stress of 24,000 lb. sq. in. was 60% of the yield stress of ASTM designation; A15–52T. In order to provide additional watertightness with less reliance on the surrounding rock to take stress, steel stresses were appropriately reduced in the vicinity of tunnel portals, adits, gate shafts, surge tanks, and wherever tunnel sections were required to deviate from a circular interior shape. Except for the prestressing of concrete lining which had occurred by coincidence with the pressure grouting performed to fill voids behind the lining, the Bureau to date had not constructed any prestressed concrete tunnel lining. That method would be considered, however, if safety and economy were indicated.

The difference in the material and labour economies in different parts of the world would no doubt continue to have considerable influence on the design and construction of tunnels, shafts, and underground hydroelectric plants.

Mr F. H. Lippold, of Denver, Colorado, said Dr Jaeger's discussion on pressure grouting was particularly interesting.

The possibility of external pressure buckling the thin steel lining must be given serious consideration in design of penstock and outlet works pressure tunnels on Bureau of Reclamation projects. The Bureau had three installations where minor buckling had occurred prior to putting the tunnels into operation. At two power plants, Alcova (Wyoming) and Palisades (Idaho), recessed one-way valves were being installed in the penstock tunnels. At the third, Folsom power plant (California), the external pressure in the penstock tunnel was temporary and was caused by backfill grouting operations. To avoid further buckling, a ¼-in.-dia. relief hole was drilled between each stiffener ring to relieve the temporary pressure and was welded shut after grouting had been completed. The efficiency of the one-way valves after long periods of use was not known t that time. The Bureau of Reclamation was now designing steel liners with stiffener rings with sufficiently heavy plate to withstand the anticipated external pressure under operating conditions. That procedure probably would not be economical for large-diameter tunnels under high external pressures, unless better methods of anchoring the plate to the surrounding concrete were developed.

Methods and pressures used in grouting a pressure tunnel varied greatly. Backfill grouting between the concrete and the surrounding rock was always done at low pressures, seldom more than 30 lb. sq. in. Where nternal pressures were not too high and the surrounding rock was of good quality, that might be all that was required. In suitable rock, low-pressure backfill grouting and perhaps some high-pressure rock grouting might allow a material reduction in the amount of reinforcing steel used n the concrete lining. Whether to reduce the reinforcing steel and thoroughly grout in such cases was an economic consideration. In poor rock sections, of course, grouting could not be substituted for reinforcing steel.

In many tunnels, pressure grouting the surrounding rock was highly desirable. By using a packer to keep the direct grouting pressure 10 ft or more from the lining, a pressure of twice the internal pressure could be afely used. In smaller-diameter tunnels that ratio could be increased. The Bureau had performed only low-pressure backfill grouting where steel-liner plate was used, but experience suggested that temporary relief holes through the lining might protect against buckling should high-pressure grouting be required.

Mr Otto Frey-Baer, of Baden, Switzerland, stated that the static problems encountered in calculating tunnel linings, shafts, and underground power stations could only be solved if the behaviour of the rock was known. The few tests and measurements of deformation carried out in unnels and test caverns had helped a great deal in procuring data for the alculations. It was, however, necessary to carry out further load tests in order to obtain sufficient evidence. On one hand the tests showed that n spite of the large variety in rock formations, their elastic behaviour did not vary as much as expected. On the other hand it had been observed hat the time factor was most important. With regard to the capacity of reeping and the resulting rock pressure, exact measurements in pressure unnels and large underground power stations were not available. Tests nd measurements should, therefore, also be extended in that direction.

Table 1 of the Paper showed very clearly that tests carried out in rock were interesting and justifiable from the economic point of view. The imensions of the lining of the pressure shafts at Innertkirchen, Handeck I, and Lyse had been determined by tests. The theoretical tensile stress n the steel for those shafts, without considering the participation of the ock in bearing the load, was 3,500 to 4,000 kg/sq. cm. For the other power stations, where no test results were available, the tensile stress was admitted to be less than 3,000 kg, i.e., a considerably higher weight in lining was necessary.

Tests had been carried out recently in the pressure shaft (penstock) at Rothenbrunnen in the Zervreila power scheme. The following results were obtained :—

Rock ; schists, inner diameter about 3 m.
Test shaft ; diameter 2·10 m, length 16·00 m.
Thickness of lining ; 16 mm.
Hydrostatic pressure inside the shaft ; maximum 70 kg/sq. cm.
Tensile stress in the steel ; 1,800 kg/sq. cm.
Compression stress in the concrete ; 70 kg/sq. cm.
Modulus of elasticity of the rock ; 90,000 kg/sq. cm.

On the basis of those results the thickness of the lining in the lower part of the shaft was set at 18 mm according to a theoretical stress of 4,100 kg/sq. cm., without considering the participation of the rock in bearing the load. Obviously, with that design it was essential to obtain a loose bond between rock, concrete, and steel. None of those three materials was more important than the other ; the construction was as strong as its weakest part. However, the great importance of concrete for assuring contact between rock and steel for the transmission of pressure must be emphasized. In order to provide against voids, Prepakt concrete would be used at Rothenbrunnen.

The excavation of large underground power stations was now no longer an unsolvable problem. The bearings of the machines must be adjusted with a precision up to fractions of a millimetre. Therefore, rock deformations which influenced the bearings might seriously affect operation of the plant. It was essential to take up the pressure of the rock entirely in the lining, in order to avoid such deformations. In connexion with that problem Mr Frey-Baer felt that the experiments formerly made in large railway tunnels were not given enough consideration, although mining engineers were still investigating them closely.

Rock pressure acted in every kind of rock. It would be wrong to assume that it did not exist in sound rock. In the new state of stress caused by the excavation the rock pressure developed only after a certain time on account of the creeping capacity of the rock (plastic deformation). Plenty of examples could be mentioned to illustrate that phenomenon. If the Figures in the Paper showing the underground power stations were compared, it was striking to note the various forms and linings chosen. The cross-sections of Swedish and Norwegian plants without linings would increase that variety. It would be interesting to collect the various experiences in that field and to discuss, in a later meeting, the problems arising from rock deformation in underground-power-station construction.

Mr O. M. Ferraz, of Paulo Afonso, Bahia, Brazil, considered that Dr Jaeger's Paper was in fact a symposium of the underground power plants actually constructed to date. It seemed to Mr Ferraz that they had a sixth type of power plant in Paulo Afonso. The system there was a very short one (100-m penstock and 180-m tail-race tunnel) but, owing to the variation of the river level (about 36-m) and the downstream side of the plant, the construction of a tail-race surge tank was unavoidable. Mr Ferraz agreed with Dr Jaeger that for the rock, even if it was fissured and had given a good performance in the pressure tests (as was the case at Paulo Afonso), the most important thing was to take care of the external pressure, especially where the shaft was of longer diameter (4·80 m) and the lining plates of small thickness.

The Paulo Afonso shafts were lined with steel plates (in accordance with ASTM A 113 specifications), which took one-third of the water (hydrostatic) pressure. Three thicknesses of plate were used ; $\frac{1}{4}$ in., $\frac{3}{8}$ in., and $\frac{5}{8}$ in. The concrete lining was 50 cm thick. Asbestos-cement conduits (4 in. dia.) drained the seepage to the sump tank. Holes about 4 in. long had been drilled through the grouting plugs into the rock. The rock at Paulo Afonso was of medium-quality fissured granite. Tests of deformation had been carried out by the IPT of São Paulo (Instituto de Pesquizas Tecnologicas de São Paulo) and those tests were considered essential in all underground-power-plant works. The modulus of elasticity had been determined during the pressure tests on the testing shaft, and measurements of seepage had shown the grouting to be very successful. It was preferred to anchor the steel lining especially where thick plates were used (as was the case in Paulo Afonso) in order to avoid buckling when the external pressure was still low.

The design of underground power plants depended essentially on the local conditions, but Mr Ferraz thought it preferable to support the rock on the upper part of the power house by using reinforced concrete arches instead of a continuous concrete lining. To avoid the falling rock a grid of angles had been used at Paulo Afonso for the big pieces (80 cm), a steel-rod grid for the medium pieces (4 in.), and finally aluminium sheets for very small pieces and seepage. Under the lower part of the ceiling cellulose sheeting had been fitted. The arch method seemed very effective to avoid uplift, which might occur in the case of continuous concrete lining of the roof. On the side walls the rock was left bare between the columns.

A special detail used in the Paulo Afonso design was the construction of the columns with a deep anchoring in the rock in order to hold the rock wall and to avoid crumbling. The horizontal anchor rods were welded to the column skeleton. The walls could not bend because of those bracings (supports). The lower part of the cavern had been excavated just after the concreting arches, and then the columns had been conconstructed in sections, from the top to the bottom. Upon completion of the cavern excavation, and before concreting the scroll casings, the height of the column and consequently of the wall had been about 20 m and the danger of a slide produced by lateral pressure had been serious. In fact the anchored columns had made the power-house into a real retaining wall which would support all the lateral stress that the vertical load might produce. Mr Ferraz had seen two cases of slides or the beginning of slides in underground power plants caused by the load on rather poor rock.

The Author, in reply, said that much valuable information had been provided by the contributors to the Correspondence. When consideration had also been given to some additional material which had been published recently,[100–109] it was possible to reach some provisional conclusions and to give a fair summary of the present trends in the design of lined and unlined tunnels and large excavations for underground stations. Before the progress in the design of pressure tunnels and shafts and of underground power stations could be fully appreciated it was indispensable for all the available data to be published with comments by the engineers responsible for design and construction.

Whilst a number of contributors had presented first-hand information of the nature which was so desirable, others had given valuable details of special problems. For instance, Mr Harris had given interesting details of the Freyssinet prestressing method as applied to two North African tunnels and one surge shaft. The method made an interesting comparison with the one described in reference 104.

Mr Talobre had commented on the theory of Heim for hydrostatic pressure distribution in the rock at great depth and had concluded that all the available evidence showed that basic assumption to be correct. His remarks, with which Mr Frey-Baer agreed,[106] were very important, for all new developments in the design of large tunnels rested implicitly or explicitly on that theory.

Mr Talobre had also mentioned rock-strain measurements made in different countries. The theory was also confirmed by Mr Harris, Mr Ward, and Frohnholzer,[102] who applied prestressing methods, the success of which would be doubtful if the basic assumption on rock natural prestresses failed to be true. There was no information whatever that the theory of Heim was not valid at great depth.

All the results of measurements made inside test tunnels and galleries or test spheres confirmed that pressure was being transmitted from the steel shell and concrete linings to the rocks as if the rock was able to take tensile stresses up to the value of the prestresses. The ability of the rock to withstand pressure depended less on its crushing strength than on its modulus of elasticity; there was no direct relation between rock strength and rock elasticity. A test tunnel built in crushed rock of poor quality exerting high pressure against the linings had shown [107] the rock to have a high modulus of elasticity—higher than many elastic homogeneous rocks. In another tunnel (in Bavaria) fissures had developed when the tunnel traversed fissured gneiss with granite. The distinction between rock strength and rock elasticity was important; so was the fact that crushed rock developing pressure against a tunnel lining might have a satisfactory modulus of elasticity.

According to Mr Lawton, the pressure tests at Kemano had been carried out with a pressure $p = 3{,}600$ lb/sq. in. (253 kg/cm^2) equivalent to 292% of the dynamic pressure to be expected in the pressure shaft. That figure was characteristic of the high compression stresses which a prestressed rock in situ could withstand without suffering from the tensile stress components developing in a circumferential direction. Tests had also been carried out inside two galleries near Fionnay without lining and with a gunite lining (mentioned by Messrs Hutter and Hanimann), and at Paulo Afonso (mentioned by Mr Ferraz). Mr Knapp had concluded his remarks on the tests carried out at Cubatão power station, by stating that the test results confirmed the theoretical calculations.

Mr Frey-Baer had expressed the opinion that tests carried out in rock were justifiable from the economic point of view, for they usually resulted in an economy in the thickness of steel linings. He had pointed out that the steel lining of the Zervreila tunnel had been dimensioned for a theoretical stress of 4,100 kg/cm^2, assuming no participation of the rock in bearing the load. Such a high figure would not have been permissible without tests made on the site to get the necessary information on the rock elasticity. The importance of tests had also been stressed by Mr Lawton.

TABLE 12

Test tunnel or rock foundation	Type of tunnel	Country	Modulus of elasticity E: kg/cm^2	Type of rock	Percentage of pressure transmitted to rock
Tiefencastel	Concrete-lined	Switzerland	92,000 to 168,000	Schists of varying quality	98
Zervriela Rothenbrunnen	Steel-lined	,,	80,000 to 126,000	Hard lime-stone	62—68
Rosshaupten	Prestressed concrete	Austria	30,000 to 45,000	Sandstone	—
Kemano	Steel-lined	Canada	140,000	Igneous rock formation	64—68
Dokan	Dam foundations: seismic method	Iraq	240,000 to 530,000; 70,000 to 85,000	Hard lime-stone	—

Table 12 summarized the results of some recent measurements in test tunnels, etc. The analysis of those tests confirmed the Author's faith in the approach he had outlined in the Paper on the basis of previous research work and tests.

Further tests by Kohn,[108] had shown that the stressing of the steel lining closely followed the pattern of the rock strain in the plastic and the elastic stages. During the tests carried out in the Zervreila test chamber, a small gap ($K_o = 0{\cdot}12$–$0{\cdot}18$ mm) between steel lining and concrete lining had caused the steel stress to rise to 350–400 kg/cm^2 before any stress could be transmitted to the rock. As soon as that gap had been closed, radial deformation of the steel shell had followed closely the plastic rock deformation until the rock began to form elasticity.

The theory indicated that, soon after the steel lining transmitted pressure to the rock the concrete lining, compressed radially, should crack, owing to circumferential tensile stresses.

It had not been possible at Zervreila Rothenbrunnen to detect the point when that cracking did occur. A similar remark applied to the Rosshaupten tunnel [102] where the radial acting grout pressure used for prestressing the concrete lining (grout pressure $p' = 5{\cdot}75$ kg/cm^2) had been insufficient to cause the outer concrete ring to be fissured as foreseen. It was likely that the concrete showed some plasticity.

According to Knapp, tests made in Brazil had shown the point when rock began to fissure (see Fig. 45).

Tests like those mentioned not only confirmed the general assumptions on which the design and the calculation of modern tunnels and shafts were based, but a more detailed analysis showed the stresses in the steel to vary with the real strain in plastic and in elastic or cracked rock. It appeared that stress and strain transmission also occurred in crushed rock of poor quality.

The possibilities of a thin gap being formed between the steel lining and the concrete lining (see Mr Colebatch's comments on p. 569) or between concrete lining and rock or even of cavities in the concrete were major problems because of the danger of locally overstressing the steel lining. Various contributors favoured the use of Prepakt concrete [108] to overcome that danger which was obviously non-existent in rock developing pressure against the lining.

In the oral discussion, Mr Guthrie Brown had mentioned an ordinary concrete-lined tunnel in the North of Scotland with diameter of 15 ft operating under a head of 600 ft and relying mainly upon the effect of pressure grouting for rock stability. He had pointed out that at the time when the tunnel had been designed, steel was scarce. The Author had found, during a personal enquiry on the subject, that many engineers would nowadays whole-heartedly confirm Mr Guthrie Brown's design. There was a trend towards keeping tunnels without lining up to about 700 ft pressure, if the rock was sound. The rock was supposed to take the strain, assuming the overburden to be sufficient. As already mentioned, at Kemano the rock was shown to take 1,660 ft (508 m) of pressure (behind the steel lining). That figure obviously proved the ability of sound rock to absorb high hydraulic pressures.

Reinforced concrete linings were used to a less extent than years ago and the modern technique relied rather on careful pressure grouting in stages. Steel lining was recommended only for higher pressures and for watertightness.

The bolder attitude in relying upon high stresses in rock was justified if the hypothesis of Heim was admitted to be correct. A bolder attitude was also noticeable concerning the necessary rock cover to protect the tunnel. The minimum depth of rock cover had in some recent designs been reduced to values as low as 0·735 times the static head in London Clay[104] or even as low as about 0·5 in sound igneous rock formation (Canadian hydro-electric power tunnel). In those cases, the design relied—after careful study of local conditions—on the validity of the rule of Heim which showed the minimum overburden in rock to be p/W_r (when p denoted the water pressure and W_r the specific weight of the rock).

The rule of the "minimum rock cover thickness" should not be accepted with the same optimism in all cases. Much depended on the quality of the rock, and the contour lines on the ground surface, and the imperviousness of the rock. A tunnel under a nearly horizontal plain might be safer in spite of a small rock cover than a tunnel parallel to a valley with sloping sides, showing greater rock cover.

Similarly the problem of water seepage from possible small rock fissures in spite of careful grouting would also have to be studied considering all the local topographical and geological conditions. There were cases where there was no possibility of water seeping from the tunnel in large quantities because there was no possible line of water escape to form a spring somewhere.

Mr Guthrie Brown had mentioned that he used a bitumen paint to seal small cracks of the concrete lining, whereas at Tiefencastel gunite with steel wire mesh reinforcement (calculated tensile stress $\sigma = 2{,}000$ kg/cm^2, neglecting any rock support) had been used. Tests made inside a test tunnel at Fionnay had shown a gunite lining to reduce the water leakage from 0·0246 to 0·0062 gal/min/sq. yd.

All possible escape routes for water seeping from the pressure tunnel or from the pressure shafts towards the underground machine hall or towards the tail-water tunnel had to be carefully sealed by grouting and draining. Careful consideration should be given to the protection of the underground power house against seepage from the pressure shafts.

The problems of grouting and draining had been carefully discussed by several contributors. The information on grouting pressures given by Messrs Lippold and Lawton and by Mr Banks in the oral discussion afforded interesting comparisons with that given by Mr Guthrie Brown.

According to Messrs Hutter and Hanimann the dynamic pressure on the Fionnay (Casanna) steel-lined tunnel was 290 lb/sq. in. (20·5 kg/cm^2) and the grout pressure adopted was 568 lb/sq. in. (40 kg/cm^2) or twice the dynamic pressure. An important remark by the contributors showed that they intended to repeat the high-pressure grouting.

The grout pressures used for prestressing tunnels were $p' = 65$ lb/sq. in. (4·6 kg/cm^2) for tunnels in North Africa (Freyssinet method described by Harris) and $p' = 80$ lb/sq. in. (5·75 kg/cm^2) for the Rosshaupten tunnel (Kieser method).

All the contributors who had dealt with the problem of grouting agreed with the indications given in the Paper. The Author felt that the rules of thumb relating the grout pressure to the rock cover were rather conservative and that with the exception of few special cases, the grout pressure should be related to the internal water pressure in the tunnel, disregarding some of the over-cautious rules concerning the rock cover.

Mr Thornton in the oral discussion had suggested that the effect of the grouting on the modulus of elasticity E should be considered. In fact most of the values for E had been obtained after some grouting (at least backfill grouting) had been done. The Author fully agreed with Mr Thornton's suggestion.

A problem which had never been dealt with was the effect of grouting of crushed rock before lining and the effect on rock plasticity and on rock pressure. Was there any possibility of using grouting for stabilizing cracked rock before blasting the tunnel or for avoiding the crushed rock beginning to creep before lining?

It was the Author's belief that creeping of crushed rock could sometimes be avoided at an early stage by using small radial pressure or that plastic rock deformations could be stopped by applying radial compression to the rock. He suggested that the arches forming the roof of underground power houses could be prestressed in order to develop a uniform radial pressure against the rock. The additional arch reaction on the arch abutments might help to stabilize the walls of the underground power house in cases where that was required and so spare the more costly construction of an oval or parabolic power house cross-section.

The Author had previously mentioned the trend towards keeping the pressure tunnels without steel lining up to a pressure of 600–700 ft and even more. One of the reasons for that bolder design technique was the awareness of the danger for thin steel linings to buckle under the action of external water pressure. The problem had been mentioned by Messrs Brickey and Lippold, and several contributors had dealt with it at some length.

Mr Knapp had compared his findings on the stability of compressed shells, with the first theory published by Mr Armstutz.* Mr Knapp had assumed that there was no gap between steel linings and concrete. He showed how the prestressing of the thin steel shell with grout pressure considerably increased the stability of the shell against buckling. A gap, even a very thin one, immediately decreased the buckling strength of the lining.

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* Reference is made here to item 41 in the Bibliography on p. 172. It should be noted, however, that Armstutz has published a correction of one of his initial assumptions, introducing prestressing of the steel liner.

The Author was fully aware of that fact. Whatever the cause of those gaps (bad conditions during the concreting, voids in the concrete, shrinkage of concrete, evaporation of excess water, temperature drop) their occurrence could not be denied.

Several contributors proposed to use Prepakt instead of ordinary concrete, which practically eliminated the danger of a gap, except for the effect of low temperatures, which might be dangerous immediately after emptying a steel-line conduit which had previously been filled with cold water. Some degree of prestressing with pressure grout seemed indispensable if gaps were to be avoided.

Another approach to the problem was the drainage of the rock to avoid the building up of outside water pressure.

Referring to the numerous comments on drainage, the Author observed that, with the exception of special cases where local drainage might be required to relieve local pressure, he favoured whenever possible the solution used at Gondo and Kemano, and suggested that in some cases model tests could give valuable information on the general pattern of flow of underground water and the possible efficiency of drainage systems.

The Author had mentioned in the Paper that Mr Talobre had worked out some general rules for the design of pressure shafts. Mr Knapp had suggested another set of very useful rules for the designer, and many useful hints on design procedure had been given by other contributors.

In conclusion the Author stated that the aim of the Paper had been to summarize the present views on the subject, to tabulate the main data and facts on pressure conduits and underground power stations, to enumerate typical designs, and to provoke comment. That had been achieved. A great deal of information from many countries showed considerable agreement between engineers. There was now little doubt that the general approach to the problem as outlined by the Author in the Paper was accepted by most of the specialists.

Many points, however, still required further investigation. Real knowledge on the behaviour of rock in situ was still insufficient. Plastic creeping of the rock required further study. There was no known method of diagnosis of the rock from an engineering point of view and deciding on the safest and cheapest design convenient to a given type of rock. All efforts should be towards better and bolder techniques. Research on the best methods for rock grouting and the effect of grouting was still in its infancy. The whole problem of steel plate buckling, outside water pressure, and rock drainage was still very controversial. The discussion and correspondence on the Paper had made it clear that there was some urgency in getting more information on all those points, and it was the Author's conclusion that that could best be achieved by a permanent technical "Committee on Underground Power Stations."

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Correction: Reference 47 on p. 172 should read: E. Gigli, "Contribution to the Study of the Behaviour of Penstocks built in Rock." *Elettrotechnica*, p. 546 (Nov. 1949).

ELECTION OF ASSOCIATE MEMBERS

The Council, at their meetings on the 26th April and 24th May. 1955, in accordance with By-law 14, declared that the following had been duly elected as Associate Members :—

Abbas, Abdel Moneim.
Adamson, Alfred Ronald.
Archer, Brian, B.Sc. (*Durham*).
Athukorala, Don Gunasena, B.Sc. (Eng.) (*London*), Grad.I.C.E.
Baker, George John Watling, M.C.
Balasubramaniam, Kumarasamy, B.Sc.(Eng.) (*London*).
Barltrop, Geoffrey Ernest, Grad. I.C.E.
Bell, Hilary, B.A. (*Oxon.*).
Bickle, Bernard John.
Blanchfield, Arthur John Stuart, B.Sc.(Eng.) (*London*).
Bowles, Anthony John Hastings, B.Sc.(Eng.) (*London*).
Brittain, Patrick Bernard, Grad. I.C.E.
Buchanan, Kenneth George Gray, B.Sc. (*Edinburgh*).
Bunyan, John Edward, B.Sc.(Eng.) (*London*).
Camacho, Robert Francis, B.Sc. (*Bristol*).
Clark, John Hedderwick, B.Sc. (*Glasgow*), Stud.I.C.E.
Cleur, Charles William.
Cocks, Donald Eric.
Crosby, Algernon Bernard, B.Sc. (*Edinburgh*).
Crosier, Thomas Arnold, B.Sc.(Eng.) (*London*).
Cuthbertson, James, Grad.I.C.E.
Dawson, Henry John, M.C., B.Sc. (Eng.) (*London*).
Dias-Abeyesinghe, Annesley Frederick, B.Sc.(Eng.) (*London*), Grad. I.C.E.
Diverall, William, B.Sc. (*Glasgow*), Stud.I.C.E.
Dobson, James Francis, B.Sc.(Eng.), Grad.I.C.E.
Eagles, Norman Alexander.
Edwards, John Michael, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Elliott, Richard Desmond, B.Sc. (Eng.) (*London*), Grad.I.C.E.
Fealdman, Harold, M.A. (*Cantab.*), Grad.I.C.E.
Fernando, Edmund Carlo.
Fernando, Ratugamage Upali, B.Sc. (Eng.) (*London*).
Goonewardena, Reginald Stanley Fonseka, B.Sc.(Eng.) (*London*), Grad. I.C.E.
Gunaratnam, Sinnappah, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Gunasekera, Godfrey de Silva.
Gunawardene, Gerard Vaz, B.Sc. (Eng.) (*London*), Grad.I.C.E.
Hall, Eric George, B.Sc. (*Aberdeen*).
Hall, James Logan, Grad.I.C.E.
Hearnshaw, James John, B.Sc.(Eng.) (*London*).
Henbest, Reginald James, Grad. I.C.E.
Hill, Donald Routledge, B.Sc.(*Eng.*) (*London*).
Hodgkinson, Ian Victor, Grad.I.C.E.
Jacomb, Arthur William, B.Sc. (*Durham*), Stud.I.C.E.
James, Kenneth Charles William, B.Sc. (*Birmingham*).
Jarvis, Dennis Groom.
Kidman, Leonard Rising, B.Sc. (*Cape Town*), Grad.I.C.E.
King, Kenneth Fritz, M.A. (*Cantab.*)
Lake, Neville George, Grad.I.C.E.
Ludeman, Kenneth John, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Millett, Edgar Kay, B.E. (*New Zealand*).
Morgan, Thomas Oliver, B.Sc. (*Wales*).
O'Loughlin, Thomas Bryan, B.A., B.A.I. (*Dublin*).
Pairman, Ian Robert, B.E. (*New Zealand*).
Perera, Makevitage Graham Cecil, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Randeniya, Munitunga, Grad.I.C.E.
Reading, Edward Arthur Venables, B.Sc. (Eng.) (*London*), Grad.I.C.E.
Reddy, Purushothama Thambumalla, B.A. (*Madras*).
Roberts, Reginald George, Grad. I.C.E.
Robertson, Alexander Stuart, B.Sc. (*Glasgow*), Grad.I.C.E.
Russon, Robert George, Grad.I.C.E.

Sampson, John Brian, B.Sc. (*Wales*).

Sandford, Roy Anthony Sandford, B.Sc.(Eng.) (*London*).

Sanmuganathan, Chellappah, B.Sc. (Eng.) (*London*).

Scatcherd, John, B.Sc.(Eng.) (*London*).

Senaratne, Makewitage Johannes Francis Raphael, B.Sc.(Eng.) (*London*), Grad.I.C.E.

Seneviratne, Lokumeegodage Don, B.Sc.(Eng.) (*London*).

Senior, Alan Gordon, M.Sc. (*Leeds*), Stud.I.C.E.

Sharman, Michael John, B.Sc.(Eng.) (*London*), Stud.I.C.E.

Shields, Douglas John, B.Sc. (*Bristol*), Grad.I.C.E.

Sinnatamby, George Selvadurai, B.Sc.(Eng.) (*London*).

Smith, Gerald.

Smith, Peter, B.Sc. (*Leeds*), Grad. I.C.E.

Tasker, John Anthony Leonard, B.Sc.(Eng.) (*London*), Grad.I.C.E.

Waugh, Thomas Miller, Stud.I.C.E.

Webb, Norman Keith, B.A. (*Cantab.*), Grad.I.C.E.

Withana, Godwin Felix Patuwatha, B.Sc. (Eng.) (*London*), Grad.I.C.E.

Wright, Colin, Grad.I.C.E.

Wright, Stanley Cempson, B.Sc. (Eng.) (*London*), Grad.I.C.E.

DEATHS

It is with deep regret that intimation of the following deaths has been received.

Members

John Aldhelm Raikes Bromage, C.I.E. (E. 1917, T. 1933).
William Henry Clark (E. 1925).
Percy John Cowan (E. 1903, T. 1927).
Alfred Douglas Creer (E. 1905, T. 1919).
John Henry Wales Laverick (E. 1924).
Laurence Lee-Dunham (E. 1925, T. 1942).
William Ransom (E. 1905, T. 1922).
Francis Bartlett Richards (E. 1934).
Frederick Sanderson Robins (E. 1894, T. 1919).
William Herbert Shields, B.Sc. (E. 1897, T. 1923).

Associate Members

Ian Evelyn Allanson, M.A. (E. 1951).
William Douglas Barraclough (E. 1918).
Roland Hirst Beaumont (E. 1906).
Norman Buckley (E. 1947).
William Clifford (E. 1906).
Reginald Tom Fitzpatrick, B.Sc. (E. 1928).
Russell Taylor Haworth (E. 1952).
Percy Stewart Heaselden (E. 1920).
John Hunter (E. 1923).
Sydney William Luscher (E. 1919).
Walter Dow Mathieson (E. 1923).
Cyril Graham Smith, O.B.E. (E. 1910).
Hubert Royds Tidswell, B.Sc.(Eng.) (E. 1911).
Herbert Nicol Weldon (E. 1890).
John Everard White (E. 1938).

REPORT OF THE COUNCIL AND STATEMENT OF ACCOUNTS 1954–55

In accordance with the By-laws, the Council present the following report on the state of the Institution.

Meetings.—Eight Ordinary, three Joint, and twenty-four Divisional Meetings have been held, a total of thirty-five as compared with thirty-four in Session 1953–54. Of the two Joint Meetings one was held jointly with the Institution of Electrical Engineers at that Institution and the other with the Société des Ingénieurs Civils de France (British Section) at which this Institution acted as host.

The Joint Meeting with the Institution of Mechanical Engineers and the Institution of Electrical Engineers, inaugurated the annual Graham Clark Lecture, which the Councils of the three Institutions have established as a memorial to the late Secretary.

A list of the Papers and Lectures presented at the meetings will be found in Appendix I.

The Opening Meeting of Session 1954–55 took place on the 2nd November when Mr David M. Watson in his Presidential Address dealt firstly with the position of the civil engineer in modern life, and the ignorance of the general public of the important works he carries out and of his great contribution to the progress of civilization. He then spoke of the relationship between the engineer and the contractor, expressing satisfaction at the harmony now existing between them. Mr Watson concluded his Address by reviewing developments in public health engineering and dealt with the new problems in the purification of sewage arising from the increasing discharge of industrial wastes, radioactive substances, and detergents.

Awards for Papers.—The awards for Papers presented at Ordinary and Divisional Meetings in Session 1954–55 and the awards for Papers printed in the Proceedings with written discussion only, between January and December 1954, will be announced early next session.

The list of awards for Session 1953–54 was published in the Proceedings in January 1955 (Pt I, vol. 4, p. 29).

Conferences.—*Conference on Correlation between Calculated and Observed Stresses and Displacements in Structures.*—This Conference is to be held in September 1955, and will be reported upon in the next Annual Report.

Conference on Engineering Education.—The second of these international conferences was held in Zurich in September 1954 and was attended by representatives of the engineering societies of Western Europe and the U.S.A. (EUSEC), who have recommended that a third Conference should be held in Paris in 1957.

The Public Works and Municipal Services Congress and Exhibition.—The Council sponsored one session of this Congress, which was held in November, and the general arrangements for which were made by the Institution of Municipal Engineers; the Paper presented for discussion was: "Mechanical plant: its employment in relation to site conditions," by W. K. Laing, M.A., A.M.I.C.E., and E. U. Broadbent, B.Sc.(Eng.).

Conference of Engineering Institutions of the British Commonwealth.—The third conference in this series was held in London from the 24th May to the 4th June, 1954. The Institution of Civil Engineers, the Institution of Mechanical Engineers, and the Institution of Electrical Engineers were the hosts on this occasion, and representatives attended from Australia, Canada, India, New Zealand, Rhodesia, and South Africa. Many matters of common interest were discussed and a report has been published.*

Conference on Civil Engineering Problems in the Colonies.—The fourth conference in this series was held at the Institution from the 14th to the 18th June, 1954. The programme followed the general pattern of the previous conferences and comprised technical sessions, visits to works, and evening receptions by the Government and the Institution.

The following is a list of the Papers presented for discussion:—

"Cross River Bridge, Nigeria; Construction and Organization Problems," by K. E. Hyatt, B.Sc.(Eng.), M.I.C.E.

"Design and Construction of the Humming Bird Highway, British Honduras," by Lt-Col. E. R. Rowbotham, M.A.

"The Hydro-Electric Developments in Territories administered by the Colonial Office," by C. H. Pickworth, M.I.C.E., M.I.E.E., and E. B. Cocks, B.E., A.M.I.C.E.

"The Main Drainage of Dar-es-Salaam, Tanganyika Territory," by H. H. Dixon, M.A., M.I.C.E.

"The Maintenance of Civil Engineering Plant of the Public Works Department, Federation of Malaya," by A. B. Watson, A.M.I.Mech.E.

"The Maintenance of Civil Engineering Plant of the Public Works Department in Northern Rhodesia," by Lt-Col. S. F. Gauron, M.B.E., A.M.I.C.E., A.M.I.Mech.E.

"Design and Construction of Harbour Improvements, Freetown," by D. C. Coode, M.I.C.E.

In addition there was an opening Lecture "Geology as Applied to Civil Engineering with Special Reference to Work in the Colonies," by F. Dixey, C.M.G., O.B.E., D.Sc., F.G.S., M.I.M.M.

Round Table Conference on Highway Problems in the Colonies.—The Council, at the request of the Colonial Office, organized this second Conference at the Institution from the 21st to 23rd June, 1954, immediately following the Conference on Civil Engineering Problems in the Colonies,

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* *Chartered Civil Engineer*, March 1955, p. 38.

The Colonial Office, the Crown Agents for Oversea Governments and Administration, the Road Research Laboratory and Colonies were represented. A summary of the report of the Conference has been published.*

Publications.—The circulation figures for the "running" publications are :—

Proceedings
Part I, published 6 times a year—22,000 copies of each number
,, II, ,, 3 ,, ,, — 7,300 ,, ,, ,, ,,
,, III, ,, 3 ,, ,, — 8,400 ,, ,, ,, ,,

Chartered Civil Engineer, issued 6 times a year—21,300 copies of each number.

Geotechnique, issued 4 times a year—1,200 copies of each number.

Railway Engineering Abstracts, issued monthly—1,200 copies of each number.

The following have been published during the year :—

Proceedings of the Conference on Civil Engineering Problems in the Colonies, 1954.
General Conditions of Contract—Fourth Edition (January 1955).
Report of the Joint Committee on the Location of Underground Services.
Supplement to the Bibliography on Soil Mechanics covering the year 1953.
Supplement to the Bibliography on Prestressed Concrete covering the year 1953.

Engineering Divisions.—Each Division completed a programme of three meetings and paid visits to a number of works of engineering interest. The titles of the Papers and the names of the Authors will be found in Appendix I.

Library.—The re-organization of the upper library has been completed and additional shelving has been provided to accommodate the files of society journals and periodicals, many of which were inconveniently housed in the basement.

Amongst the many acquisitions in the course of the year mention should be made of the unique unpublished MS of John Grundy's Reports and Surveys *circa* 1740–60 purchased through the Unwin Bequest.

During the year, 557 volumes were presented to the Library and 397 were purchased, making a total on the 31st March of 69,717 volumes and 18,867 pamphlets. 4,843 applications for the loan of books were dealt with.

Local Associations.—During the year the Local Associations have continued their useful activities; brief accounts of these appear in Appendix II.

There have been some changes amongst the Honorary Secretaries.

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* *Chartered Civil Engineer*, May 1955, p. 24.

Mr R. White succeeded Mr H. M. Nelson as Honorary Secretary of the Glasgow & West of Scotland Association, and Mr J. R. Freeman took over from Mr G. C. Mander in the South-Western Association.

Awards made by the Council for Graduates' and Students' Papers presented before Local Associations in Session 1953–54, included Miller prizes to two Graduate Members and one Student Member of the Edinburgh & East of Scotland Association ; to one Graduate Member and two Student Members of the Glasgow & West of Scotland Association and to a Graduate Member in each of the Northern Counties and South-Western Associations.

The Vernon Harcourt Lecture "The siting and design of harbours," by Mr E. J. Buckton, was repeated before the Northern Counties, Southern, and the South Wales & Monmouthshire Associations.

Oversea Associations.—The 26-year-old Malayan Association has continued its activities under the Chairmanship of Mr D. I. Todman, who succeeded Mr G. M. Wheat. Mr L. D. Smith succeeded Mr A. D. Eaton as Honorary Secretary and Mr J. H. C. Shakespear continued as Honorary Treasurer. There has been a steady increase in membership, which totalled 135 at the end of February 1955. A successful Annual Dinner was held at the Selangor Club, Kuala Lumpur, in November 1954 at which the Member for Works of the Government, Mr R. B. Carey, was the principal guest. The Association Journal was published as frequently as possible and visits to engineering works were arranged at about 2-month ntervals. In January 1955 a petition was signed by the local representatives of the Institution of Civil Engineers, the Institution of Mechanical Engineers, and the Institution of Electrical Engineers, requesting that the Singapore Joint Oversea Group of the three Institutions be extended to cover the Federation of Malaya. It is proposed that the Association should continue its activities so long as the need is felt, notwithstanding such extension of the Joint Group.

The membership of the Victorian Association reached a total of 158 during the year. Two meetings of the Committee were held ; also a unction comprising a buffet supper and social evening, followed by films of the hydro-electric work of the Snowy Mountain Authority. The Chairman of the Association has been Mr B. J. Callinan, and the Honorary Secretary, Mr F. W. Green.

The informal group formed last year in Tasmania held one meeting with a discussion on "Education and the engineer."

Mr A. H. Richard, B.Sc., M.I.C.E., has been the Chairman of the Trinidad Branch of the West Indies Association, and Mr D. Watson succeeded Mr C. Dupenois as Honorary Secretary during the year. Several meetings and visits were arranged and the Annual Dinner was again held jointly with the other two Institutions. The principal guests were His Excellency The Governor, Sir Hubert Rance, G.C.M.G., G.B.E., C.B., and Lady Rance. Her Royal Highness Princess Margaret, during her visit to the Colony, opened and named the island's arterial road to the south as "The Princess Margaret Highway" and visited other major works recently completed in the Colony.

Joint Oversea Groups.—The vigour with which the activities of these youthful groups are pursued, augurs favourably for the future well-being of the profession overseas.

The Argentine Joint Oversea Group continued under the Chairmanship of Mr W. H. R. Webb, A.M.I.Mech.E., with Mr J. H. Robinson, A.M.I.Mech.E., as Honorary Secretary.

The Hong Kong Joint Oversea Group continued its activities under the Chairmanships of Mr S. E. Faber, A.F.C., B.Sc.(Eng.), A.M.I.C.E., and Mr A. W. Black, M.I.Mech.E. Mr J. J. Robson, A.M.I.C.E., continued as Secretary.

During the first year of full-scale operation of the Iraq and Persian Gulf Joint Oversea Group, six meetings were held—two each at Baghdad, Basrah, and Kuwait. Each meeting extended over two days, a Paper being read on the first day and a visit taking place on the second. The Group Committee also hope to be able to arrange for meetings to be held in Bahrein and Kirkuk.

The Singapore/Malayan Joint Oversea Group arranged during the year 17 meetings and 8 visits in Singapore and the Federation of Malaya, at which good attendances were maintained. The new Committee, elected in February 1955 by means of a postal ballot, resulted in the appointment of Mr W. G. Scott, M.I.E.E., and Mr Ng Wah Hing, M.A., Ph.D., A.M.I.C.E., to succeed Mr R. J. Hollis-Bee, M.I.C.E., and Mr W. Trafford, M.B.E., A.M.I.E.E., as Chairman and Honorary Secretary respectively. The extension of the activities of the Joint Group to cover the Federation of Malaya was the subject of an official petition to the three Institutions in London.

In the West African Joint Oversea Group Mr J. Houston Angus, M.I.E.E., succeeded Mr Alexander McDonald, B.Sc., M.I.C.E. (who was appointed Secretary of the Institution), as Chairman; and Mr E. J. Evans, B.Sc., A.M.I.C.E., was succeeded by Mr R. G. M. Bathgate, B.Sc., A.M.I.C.E., as Honorary Secretary. Five Papers, five visits to works, and two film shows were arranged, in addition to a successful first Annual

Dinner at which the Governor, Sir John Macpherson, and several Ministers were the guests of honour. A branch of the Committee was formed in the Gold Coast. Mr I. Small, B.Sc., A.M.I.C.E., acted as Honorary Secretary and visits to works and the presentation of Papers were arranged.

The Association of London Graduates and Students.—The Opening Meeting was held on the 3rd November, when the Chairman of the Association, Mr J. A. Percival, B.Sc.(Eng.), Grad.I.C.E., gave an Address on "Water Purification," the President taking the Chair.

The Vernon-Harcourt Lecture, 1955, on "The siting and design of harbours," was delivered by Mr E. J. Buckton, B.Sc.(Eng.), M.I.C.E., on the 5th January.

Three meetings were held for the reading and discussion of Graduates' and Students' Papers, and two meetings were held at which informal addresses were delivered by invited speakers on the subjects of "Aluminium as a structural material" and "Surveying methods and oxygen apparatus used on recent Himalayan expeditions." A joint meeting with the Graduate and Student sections of the Institution of Mechanical Engineers and the Institution of Electrical Engineers, took place on the 14th March, when Sir Miles Thomas, D.F.C., M.I.Mech.E., gave a talk on "Air Transportation."

Seven visits to works were arranged during the session and attendances at both these and the meetings were very satisfactory.

Social events included a Smoking Concert following one of the informal addresses, a joint dance with the Graduate and Student sections of the Institution of Mechanical Engineers and the Institution of Electrical Engineers, and the Young Trophy sporting events in which the Institution was unfortunately placed third.

Competition for the Institution Medal and Premium (London University).—The Eighth Competition for the Institution Medal and Premium (London University) 1954, was held at a meeting in the Lecture Theatre of the Institution on Wednesday, 19 May, 1954.

The President (Mr W. P. Shepherd-Barron) was in the Chair, and the following Papers were presented in competition:

"Tunnelling under compressed air in non-cohesive soils applied to a large-diameter sewer," by J. C. Joel, Stud.I.C.E. (King's College).

"The building of the central section of Stockholm underground railway," by P. Rance (City and Guilds College).

"General aspects of coal-mining in Great Britain," by P. A. Thompson (Queen Mary College).

Mr H. Shirley Smith, Mr J. H. W. Turner, and Mr R. M. Wynne-Edwards acted as Judges and, on their recommendation, the Institution

Medal and Premium of £10 were awarded to Mr J. C. Joel (King's College). The Judges gave a Commendable Mention to Mr P. Rance (City and Guilds College) and awarded him a Premium of £5.

The competition was well attended by undergraduates of the various London colleges and the discussions which ensued were of a high order.

Competition for the Institution Medal and Premium (Local Associations).—This Competition was held in the Brunel Room at the Institution on the 21st October, 1954, with the President (Mr W. P. Shepherd-Barron) in the Chair, when the following Papers were presented :

"Concrete work at Upper Glendevon Reservoir," by B. N. Harvey, Grad.I.C.E. (Edinburgh & East of Scotland Association).

"First-stage road works of a new approach road to Swansea," by M. J. Hill, Stud.I.C.E. (South Wales & Monmouthshire Association).

"Stream intakes," by W. M. Sommerville, Stud.I.C.E. (Glasgow & West of Scotland Association).

The Judges appointed by the Council were Mr W. K. Wallace, Dr A. W. Skempton, and Mr C. B. Townend and the winner was declared to be Mr B. N. Harvey, Grad.I.C.E., of the Edinburgh & East of Scotland Association.

Examinations.—The numbers of candidates who applied for admission through the Institution and attended the Common Preliminary Examination conducted by the Engineering Joint Examination Board, were 99 for the October 1954 Examination and 143 for the April 1955 Examination.

The number of candidates examined in the October 1954 Institution Examination (Parts I and II) was 452, including 87 at 22 centres overseas. For the April 1955 Examination there were 562 candidates, including 87 at 16 centres overseas. In October 1954 and April 1955, 917 candidates were examined for the Professional Interview including 230 at 42 centres overseas.

Bayliss Prize.—For the April 1954 Examination the Bayliss Prize was awarded to Mr Arthur Mascall, Stud.I.C.E., of South Shields, and that for the October 1954 Examination to Mr C. J. Whitlock of London, an approved candidate for election.

Research.—No new committees have been appointed during the year, but four, having completed their tasks, have been dissolved. The following is a summary of the work of the Research Committee :—

Abstracting.—Replies to the questionnaire concerning the possible publication of "Civil Engineering Abstracts," which was sent to 1,000 corporate members selected at random, have been analysed. Only 359 replies were received, and the analysis has shown that approximately 10% of members could be expected to take the abstracts if published. Before making its final report, however, the Sub-Committee is conducting further enquiries to see what possible demand for "Civil Engineering Abstracts" there might be from other directions and to try to get a clearer idea of probable costs.

Coast Protection.—The Second Interim Report of this Committee has been published in *Chartered Civil Engineer.* This report draws attention to researches required to give a better understanding of the factors involved in the design of coast protection works. Copies of the Report have been presented to the two Government Departmental Committees, on Sea Defence Works and on Oceanography, set up as the result of recommendations in the Report of the Departmental Committee on Coastal Flooding (the Waverley Report).

Compressed Air.—Publication of the Diving and Compressed Air Special Regulations, by the Ministry of Labour and National Service, is still awaited.

Erosion of Earthen Banks.—As a result of recommendations contained in the Waverley Report it appears likely that some investigations on earthen banks will be undertaken on behalf of a Departmental Committee and it may be unnecessary to maintain the Institution Committee.

Hydraulics.—The three sub-committees appointed to prepare reviews of recent developments in various branches of hydraulics have completed their task and a "Review of Recent Developments in Hydraulics" is to be published in the Proceedings. Separate copies will be available.

Location of Underground Services.—The revision of the 1946 Report of the Joint Committee set up in conjunction with the Institution of Municipal Engineers having been completed, and a new edition of the Report published, the Committee has been dissolved.

Mechanical Properties of Aggregates.—"A review of the methods of testing aggregates for structures other than roads or airfields" has been published in the Proceedings, Part I, May 1955 (separate copies are available), and the Committee has been dissolved.

Mining Subsidence—The Committee hopes, in time, to prepare a report giving guidance on the best methods of design and construction which can be adopted to alleviate damage due to subsidence, and to this end four sub-committees have been set up to collect information regarding bridges, roads, structures, and public utilities.

Performance of Concrete Mixing Machinery.—In view of the reluctance of manufacturers to co-operate with the Committee and the fact that some of them are actively co-operating with the Road Research Laboratory, where work is being done on lines formulated by the Committee, it has been decided that this Committee has fulfilled its purpose as far as possible, and it has been dissolved.

Pile-Driving.—No more data of test results have been received and there is nothing further to report.

Prestressed Concrete Development.—Work has commenced on a review of developments in recent years. The third Supplement to the Bibliography on Prestressed Concrete, for the year 1953, has been published and further supplements for 1954 and 1955 are being prepared.

Quality of Concrete in the Field.—The "Report on the quality of concrete in the field" has been published in the Proceedings, Part I, May 1955 (separate copies are available), and the Committee has been dissolved.

Sea Action.—The 19th Interim Report (2nd Edition) has been reprinted, and a short supplement dealing with the Series XIII timber specimens which have been undergoing tests at Colombo, Singapore, and Upnor since 1949, has been printed for issue with the reprinted Report.

Soil Mechanics and Foundations.—The sixth Supplement to the Bibliography on Soil Mechanics, for the year 1953, has been published and further supplements for 1954 and 1955 are being prepared.

Vibrated Concrete.—A final draft report has been prepared and circulated for approval by the Committee. The report should be published in the course of the next few months.

British Standards Institution.—During the year a number of British Standards have been produced by committees on which the Institution has been represented, and more members have accepted invitations to serve the Institution on new technical committees of the British Standards Institution.

Codes of Practice.—Civil Engineering Codes of Practice No. 3: Earthworks, and No. 6: Traffic Bearing Structures—Pavings, have been issued for comment by the British Standards Institution under the new arrangement for dealing with Codes of Practice, and the comments received are being collated for consideration of the committees concerned.

Public Relations.—Three Christmas Lectures for Boys were arranged at the Institution.

"Speed and the motorway," by Lt-Col. E. W. W. Richards, A.M.I.C.E.

"The wonders of big bridges," by H. Shirley Smith, O.B.E., B.Sc., M.I.C.E.

"The story of communications in tropical Africa," by R. W. Taylor, C.M.G., B.Sc., M.I.C.E.

During the year the Council approved the preparation of an exhibit for display at careers exhibitions and the like. The exhibit was professionally designed and consists of one introductory panel and four others under the respective headings of Power, Transport and Communications, Public Health, and General Structures. The panels are designed to be readily transported and erected, and are complete with lighting. Each panel carries a written legend in addition to photographs of typical civil engineering works. The mounting of the photographs is such that new photographs can be used as necessary to keep the exhibit up-to-date. The exhibit has already been shown at two Exhibitions and will be available on loan.

The Council has given much thought to making the general public more aware of the work of the civil engineer, and some progress has been made. The position, however, is still unsatisfactory and the Council is continuing its activities in this direction.

Relations with the Institution of Mechanical Engineers and the Institution of Electrical Engineers.—The closest association has continued to exist between the three major Institutions, particularly in educational matters, since engineering education plays an increasingly important part in the activities of the three Institutions. The Council of the Institution of Mechanical Engineers has now become a third partner in the Part I Examination, and it is hoped that other engineering Institutions will participate. Reference was made in the Council's last Annual Report to the representations then being made to the Minister of Education on the question of higher technological education in the United Kingdom, and to the Colonial Secretary in relation to the development of engineering education in the colonies. These representations are continuing.

The Engineers' Guild.—A Joint Committee has been set up, consisting of two representatives from each of the Institutions of Civil, Mechanical, and Electrical Engineers and the Engineers' Guild, with the Secretaries, to consider matters of common concern and to report as necessary to the respective Councils.

Accounts.—The accounts for the year ending the 31st March, 1955, which have been duly audited, are detailed in Appendix IV of this Report and may be summarized briefly as follows :—

The Total Income for the year amounted to . . . . (as compared with £94,759 last year) including £409 for Income Tax recovered. Subscriptions, Entrance Fees, and Examination Fees totalled £91,741 (as compared with £89,645 last year), and Dividends and Interest received amounted to £2,377 (as compared with £2,374 last year).	£97,327
The Total Expenditure charged against the year's Income amounted to . . . . (as compared with £91,276 last year).	£89,121
The General Revenue Account therefore results in a credit balance on the year of	£8,206
The General Revenue Account Surplus amounted to £35,102 on the 31st March, 1955, made up as follows :—	
Balance brought forward 1st April, 1954 . . .	£26,896
Add Surplus for year to 31st March, 1955 . . .	£8,206
	£35,102

Cash at Bankers and in hand amounted to £54,516 (compared with £43,597 last year) at the close of the financial year, owing to the receipt, as in past years, of a substantial proportion of the current subscriptions during the first quarter. This balance is required to finance expenditure during the remainder of the year.

The actual expenditure during the year on "Publications Account" amounted to £38,439 (compared with £37,542 last year), of which £26,412 represented the cost of the Proceedings, etc. This expenditure was relieved by credits for advertisements, sales, etc., of £20,981 (against £18,362 last year), leaving the net expenditure for the year at £17,458 (compared with £19,180).

The Repairs and Renewals Reserve credit balance has been reduced by £1,282 during the year, viz. from £3,745 to £2,463.

On Trust Funds Income Account there was received a total of £2,247 and the expenditure amounted to £1,428.

During the year, sums of £24 in interest and £17 for sales of reports, etc., have been credited to the research into the Deterioration of Structures exposed to Sea Action.

Prizes.—A Charles Hawksley Prize of £130 for the 1954 Competition was awarded to Mr J. S. Shipway, Grad.I.C.E., and consolation prizes to Mr D. E. Key (£100), Grad.I.C.E., Mr O. Cundall (£75), B.Sc.Tech., A.M.I.C.E., and Mr R. J. Amblin (£75), A.M.I.C.E.

Fellowships and Scholarships.—The Council awarded the Culmann Travelling Fellowship for 1954–55 to Mr H. K. Goodger, B.Sc., A.M.I.C.E., to carry out studies in Western Europe on the present trend of design and practice in the execution of engineering works of reinforced concrete and steelwork construction.

The 1954–55 Radley Research Studentship was awarded to Mr H. Kierulf, B.Sc.(Eng.), Grad.I.C.E., to enable him to attend a post-graduate course on concrete technology at the Imperial College.

Rockefeller Foundation Bursaries in Public Health Engineering.—In the final year of the initial three-year period for which the Institution agreed to administer funds for these bursaries, eleven more awards were made, one to a research student and the others for post-graduate studies. Altogether, of fifty-four applicants, twenty-eight have been successful in obtaining bursaries; six of these were for research. The successful applicants have come from many parts of the world: Australia, the British Isles, Gold Coast, India, Malaya, Mauritius, Nigeria, Norway, and South Africa.

The Institution has consented to continue this work and further funds have been allocated by the Rockefeller Foundation to enable the scheme to carry on until 1960.

Conversazione.—The Conversazione was held at the Institution on the evening of the 24th June, 1954, when 1,375 members, guests, and ladies were present. Strawberries and cream made a welcome reappearance, and dancing was enjoyed in the Great Hall.

Graduates' and Students' Conversazione.—A conversazione for Graduates and Students was held for the first time on the 22nd October, 1954, and was attended by 478 Graduates and Students and their ladies. Models were exhibited and there was dancing in the Great Hall.

Annual Dinner.—The Annual Dinner of the Institution was held on the 21st April, 1955, at the Dorchester Hotel, when the Rt Hon. the Viscount Waverley proposed the toast of the Institution and Mr Lennox Boyd, Secretary of State for the Colonies, replied to the toast of the guests.

Nominations and Appointments.—The following appointments have been made or renewed by the Council during the year :

Sir William Halcrow	British National Committee, World Power Conference.
Mr R. M. Wynne-Edwards, D.S.O., O.B.E., M.C., M.A. Dr H. Q. Golder	British National Committee, International Society of Soil Mechanics and Foundation Engineering.
Sir Arthur Whitaker, K.C.B., M.Eng.	British Section, Permanent International Association of Navigation Congresses.
Mr Ralph Freeman, C.B.E., M.A. Mr L. Scott White, O.B.E.	National Consultative Council, Ministry of Works.
Mr J. N. Peirce Mr Ralph Freeman, C.B.E., M.A. (Deputy)	Tribunal of Appeal under the London Building Acts, 1930–1939.
Mr A. S. Quartermaine, C.B.E., M.C., B.Sc.	Executive Committee of E.U.S.E.C.
Mr W. P. Shepherd-Barron, M.C., T.D., LL.D.	Science Advisory Committee, British Council.
Professor A. G. Pugsley, O.B.E., D.Sc., F.R.S.	General Board of the National Physical Laboratory.
Mr M. T. Tudsbery, C.B.E.	Governing Body, Imperial College, University of London.
Mr C. H. Stanger	Joint Committee on Materials and their Testing.
Mr F. S. Snow, O.B.E.	Concrete Technology Advisory Committee.
Mr A. C. Hartley, C.B.E., B.Sc.	General Council, British Standards Institution.
Mr A. S. Quartermaine, C.B.E., M.C., B.Sc. Mr R. W. Foxlee, C.M.G., C.B.E.	Engineering Divisional Council, British Standards Institution.

Mr J. E. Swindlehurst, O.B.E., M.A.	Building Divisional Council, British Standards Institution.
Mr Arthur Floyd, C.B.E., B.Sc. Mr H. J. B. Harding, B.Sc. Mr H. Shirley Smith, O.B.E., B.Sc. Mr W. E. Blizard, B.Sc.	Codes of Practice for Civil Engineering, British Standards Institution.
Mr J. N. Peirce Mr F. S. Snow, O.B.E.	Codes of Practice Committee for Building, British Standards Institution.
Mr W. P. Shepherd-Barron, M.C., T.D., LL.D.	General Committee, Parliamentary and Scientific Committee.
Dr W. H. Glanville, C.B., C.B.E.	Advisory Committee, Town Planning Institute, Rees Jeffreys Trust Fund.
Mr J. A. Banks, O.B.E.	General Conditions of Contract Joint Committee.
Dr W. H. Glanville, C.B., C.B.E. Mr G. M. McNaughton, C.B., B.Sc.	Board of the Nuclear Energy Conference.
Sir Arthur Whitaker, K.C.B., M.Eng. Sir George Fretwell, K.B.E., C.B. Sir Hubert Walker, C.B.E.	Joint Overseas Activities Committee.
Mr R. W. Foxlee, C.M.G., C.B.E. Mr H. Shirley Smith, O.B.E., B.Sc. Mr J. H. W. Turner, B.Sc.	Board of the Professional Engineers Appointments Bureau.
Professor A. J. S. Pippard, M.B.E., D.Sc., F.R.S.	Consultative Member of Council to the Association of London Graduates and Students, I.C.E.
Mr W. E. Doran, O.B.E., B.A., B.A.I.	Standing Joint Committee, Higher National Certificates in Civil Engineering.
Professor R. J. Cornish, M.Sc.	Engineering Joint Examination Board.
Mr S. G. Barrett	Joint Committee for Joint Part I Examination.
Mr J. H. W. Turner, B.Sc.	Standing Joint Committee for Training of Pupils with Civil Engineering Contractors.

Mr A. S. Quartermaine, C.B.E., M.C., B.Sc. Mr W. K. Wallace, C.B.E.	Joint Committee of the Inst.C.E., the Inst.Mech.E., the Inst.Elec.E., and the Engineers' Guild.

Professor A. L. L. Baker, B.Sc.Tech., represented the Institution at a meeting of the Comité Européen du Béton, organized by the Chambre Syndicale des Constructeurs en Ciment Armé de France et d'Union Français, held in Paris in May 1954.

Mr W. E. Doran, O.B.E., B.A., B.A.I., and Mr Jack Duvivier, B.Sc., have been nominated members of an Advisory Committee which has been set up by the Ministry of Agriculture and Fisheries to co-ordinate researches designed to improve sea defences in the North Sea.

Professor A. W. Skempton, D.Sc., represented the Institution at a Symposium on High Flats, which was held at the Royal Institute of British Architects on the 15th February, 1955.

International Engineering Congresses.—The British National Committee of the International Society of Soil Mechanics and Foundation Engineering has appointed an Organizing Committee, under the Chairmanship of Dr W. H. Glanville, C.B., C.B.E., to make arrangements for the Fourth International Conference which is to be held in London in August 1957.

The Permanent International Commission of the Permanent International Association of Navigation Congresses accepted an invitation, made on the initiative of the British National Committee, to hold the XIXth Congress in London in July 1957. The Rt Hon. the Viscount Waverley, P.C., G.C.B., G.C.S.I., F.R.S., has accepted the Presidency of the British Organizing Commission for the Congress.

The British National Committee of the International Commission on Irrigation and Drainage arranged for full participation by this country in the second International Congress held in April 1954 in Algiers.

British Nuclear Engineering Conference.—Members will be interested in the formation of this Conference, whose Board consists of two members each, plus the Secretaries, from the Institutions of Civil, Mechanical, and Electrical Engineers, the Institute of Physics, and the Institution of Chemical Engineers. Meetings will be held (which all members of these organizations will be free to attend), Papers presented and discussed, and a Journal published on all matters pertaining to nuclear engineering. Since the highest capital costs are represented in the civil engineering works, it seems appropriate that the Conference will be serviced by this, the oldest engineering institution, and that the meetings will be held in our handsome building in the heart of Westminster.

The Roll.—The Roll of the Institution on the 31st March, 1955, stood at 20,706, the changes which have taken place in it during the year ended on that date being shown in the Table given below :

	1st April, 1953, to 31st March, 1954							1st April, 1954, to 31st March, 1955						
	Honorary Members	Members	Associate Members	Associates	Graduates	Students	Totals	Honorary Members	Members	Associate Members	Associates	Graduates	Students	Totals
Numbers at commencement	16	2390	10188	34	1517	5025	19170	16	2413	10660	33	2502	4350	19974
Transfers : Associate Members to Members	..	+106	−106	..	..	..		..	+114	−114	..	..	..	
Elections . .	..	5	707	..	..	..	+2636	..	8	713	3	..	..	+2426
Admissions .	..	..	..	..	1093	821		..	..	..		915	759	
Restored to Roll	..	..	6	..	2	2		..	2	20	..	1	5	
Deceased . .	..	59	61	..	..	4	−1832	..	61	66	2	2	4	−1694
Resigned . .	..	23	44	..	2	51		..	16	54	..	11	41	
Erased . . .	..	6	29	1	..	73		..	4	35	..	1	48	
Elected Associate Members .	..	..	..	..	103	178		..	..	..	..	268	89	
Admitted as Graduates .	..	..	..	..	..	689		..	..	..	..	..	554	
Removed as over-age .	..	..	..	..	..	496		..	..	..	..	..	430	
Failed to complete : Transfer . .	..	..	..	..	..	..		..	..	..	..	..	..	
Election . ,	..	..	1	..	..	..		..	..	1	..	..	..	
Admission .	..	..	..	..	5	7		..	..	..	..	1	6	
							+804							+732
Numbers at termination .	16	2413	10660	33	2502	4350	19974	16	2456	11123	34	3135	3942	20706

The full list of deaths and resignations is given in Appendix III, and the Council record with especial regret the death of Sir Ernest Albert Seymour Bell, C.I.E., F.C.H. (former Member of Council).

Acquisitions.—The Council have accepted, on behalf of the Institution, Telford and Stevenson Gold Medals awarded to the late Mr G. A. Hobson, M.I.C.E., ten prize books awarded with the Telford Medal, and a walnut bookcase, presented by his daughter, Miss D. F. Hobson.

The contents of his technical library and two bookcases, were presented by Sir Richard A. S. Redmayne, K.C.B., M.Sc., Past-President.

A memorial volume entitled "The Water Supply of Greater London," by the late Dr H. W. Dickinson, M.I.C.E., was presented by the Newcomen Society.

A roll-top desk was bequeathed by Mr E. Graham Clark, the late Secretary of the Institution, which he wished should be retained by the Institution.

The sum of £3,000 (South African currency) was bequeathed to the Institution by the late Mrs E. M. Harvey "for the absolute use and benefit of the Civil Engineering branch" of the Institution.

The Secretaryship.—The new Secretary, Mr Alexander McDonald, B.Sc., M.I.C.E., took up his duties on the 28th June, 1954. The sudden death of his widely-respected predecessor, Mr E. Graham Clark, C.B.E., M.C., M.Sc., M.I.C.E., in April, precluded any handing over, and it might almost be said that another era in the 136-year-old tradition of the Institution had ended. New and wider spheres of influence are already coming within the purview of a hard-pressed secretariat, a fact of which the profession is not unappreciative, and modest increases in establishment are inevitable if the Institution and its elected Council are to fulfil their proper functions for a steadily increasing membership.

APPENDIX I

ORDINARY MEETINGS

Presidential Address of Mr David M. Watson, B.Sc.
"Present trends in the design of pressure tunnels and shafts for underground hydro-electric power stations," by Charles Jaeger, Dr ès Sc. Techn.
The Unwin Lecture on "Experimental science and civil engineering research," by F. M. Lea, C.B.E., D.Sc.
"New Northam Bridge, Southampton," by F. L. Wooldridge, M.I.C.E., John Cuerel, B.Sc., M.I.C.E., and K. R. Hauch, O.B.E., B.Sc.
"Investigations into the design of pressure tunnels in London clay," by Fred Tattersall, M.Sc., M.I.C.E., T. R. M. Wakeling, M.Sc.(Eng.), A.M.I.C.E., and W. H. Ward, B.Sc.(Eng.), A.M.I.C.E.
"Geophysical methods of exploration and their application to civil engineering problems," by Jack Robertshaw, B.Sc., A.M.I.C.E., and P. D. Brown, B.Sc.(Eng.), A.M.I.C.E.
"Potentialities of the British Railways' system as a reserved roadway system," by Brigadier T. I. Lloyd, D.S.O., M.C.
The James Forrest Lecture on "Nuclear power and world energy resources," by Sir John Cockcroft, K.C.B., F.R.S.

JOINT MEETINGS

With the Institution of Mechanical Engineers and the Instituton of Electrical Engineers:
The Graham Clark Lecture on "The engineer's contribution to the conservation of natural resources," by Brig.-Gen. Sir Harold Hartley, K.C.V.O., C.B.E., M.C., M.A., D.C.L., F.R.S.
With the Institution of Electrical Engineers:
"Problems of hydro-electric design in mixed thermal-hydro-electric systems," by T. G. N. Haldane, M.A., M.I.C.E., Past-President, I.E.E., and P. L. Blackstone, T.D., M.A., M.I.E.E. (At the Institution of Electrical Engineers.)
With the British Section of the Société des Ingénieurs Civils de France:
"Tidal energy," by Professor R. Gibrat.

ENGINEERING DIVISION MEETINGS

AIRPORT DIVISION

"Some considerations of airfield pavement design," by J. A. Skinner, B.Sc.(Eng.), M.I.C.E., and F. R. Martin, B.Sc., A.M.I.C.E.

"The requirements of a civil airport as typified by London Airport," by B. E. Willett, B.Sc.(Eng.), A.M.I.C.E.
"The extension of the runway at R.N. Air Station, Belfast," by L. R. Greenaway, B.Sc.(Eng.), A.M.I.C.E., and John Callagin, A.M.I.C.E.

HYDRAULICS DIVISION

"Model experiments on the storm surge of 1953 in the Thames Estuary and the reduction of future surges," by F. H. Allen, M.A., M.A.I., A.M.I.C.E., W. A. Price, B.Sc.(Eng.), Grad.I.C.E., and Sir Claude C. Inglis, C.I.E., M.A.I., M.I.C.E., F.R.S.
"The employment of unit hydrographs to determine the flows of Irish arterial drainage channels," by J. J. O'Kelly, B.E., A.M.I.C.E.I.
"Some hydraulic aspects of sewerage and sewage disposal," by C. B. Townend, C.B.E., B.Sc., M.I.C.E., and G. W. Wilkinson, A.M.I.C.E.

MARITIME AND WATERWAYS DIVISION

"The Queen Elizabeth II Dock, Eastham," by D. C. Milne, B.Sc., M.I.C.E.
"The design and construction of No. 8 dry dock at North Shields for Smith's Dock Co. Ltd," by T. F. Burns, M.I.C.E.
"Reconstruction of two quays on the river Ouse at King's Lynn," by D. V. Buck, M.I.C.E.

PUBLIC HEALTH DIVISION

"The design and construction of the handling and treatment system for liquid radioactive wastes," by W. L. Wilson, O.B.E., B.Sc., A.M.I.C.E., A.M.I.Mech.E., and
"Operational experiences with a handling and treatment system for liquid radioactive wastes," by R. H. Burns, B.Sc., F.R.I.C.
"The effect of storage on sewerage design," by C. D. C. Braine, B.Sc., M.I.C.E.
"The application of prestressed concrete to water-supply and drainage," by Professor R. H. Evans, D.Sc., M.I.C.E.

RAILWAY DIVISION

"Standardization of type and design applied to steam-locomotive shed roofs," by V. F. Beer, and
"Smoke extraction from engine sheds: an account of some model and full-scale tests," by R. L. McIlmoyle, M.I.C.E., and D. W. Peacock, B.Sc., A.M.I.Mech.E.
"Folkestone Warren landslips: investigations, 1948–50," by A. M. Muir Wood, M.A., A.M.I.C.E., and
"Folkestone Warren landslips: remedial measures, 1948–54," by N. E. V. Viner-Brady, O.B.E., M.I.C.E.
"Some notes on the half-through type plate-girder railway bridge," by P. S. A. Berridge, M.B.E., M.I.C.E., and F. M. Easton, A.M.I.C.E.

ROAD DIVISION

"Prestressed units for short-span highway bridges," by A. D. Holland, T.D., B.Sc. (Eng.), A.M.I.C.E.
"Prestressed concrete roads," by J. P. Stott, B.Sc.(Eng.), Ph.D.
"Working capacity of roads," by A. J. H. Clayton, B.Sc.(Eng.), A.M.I.C.E.

STRUCTURAL AND BUILDING DIVISION

Symposium on high buildings:
- "The social and town-planning significance of high buildings," by Sergei Kadleigh, A.R.I.B.A.
- "High building frames and foundations," by Professor A. L. L. Baker, D.Sc., M.I.C.E.
- "Foundations for high buildings," by Professor A. W. Skempton, D.Sc.(Eng.), A.M.I.C.E.
- "High buildings—the traffic and parking problems," by S. J. Chamberlain, O.B.E., D.F.C.

"The civil engineer and Britain's atomic factories," by D. R. R. Dick, B.Sc.(Eng.), M.I.C.E.

"Some steel structural frames designed on plastic theory," by D. H. Little, B.Sc.(Eng.), A.M.I.C.E., and A. Augustus Smith, B.Sc.(Eng.), A.M.I.C.E.

WORKS CONSTRUCTION DIVISION

"The reconstruction of Deptford Creek bridge," by F. M. Fuller, O.B.E., B.Sc.(Eng.), M.I.C.E., and J. N. C. Couper, B.A.

"A survey of modern concrete technique," by E. E. H. Bate, C.B.E., M.C., B.Sc. (Eng.), M.I.C.E., and D. A. Stewart, M.B.E., A.M.I.C.E.

"The use of explosives for demolitions," by D. H. Brook, B.Sc., and R. Westwater, B.Sc., Ph.D.

APPENDIX II *

GLASGOW AND WEST OF SCOTLAND ASSOCIATION

Mr Stanley D. Canvin (M), Chief Engineer and General Manager of Glasgow Corporation Water Department, was re-elected for a second term of office as Chairman in view of the celebration in 1955 of the Centenary of the Glasgow Corporation Water Undertaking.

To inaugurate the Seventieth Session of the Association, Mr J. A. Warren (M), who was the first Chairman in 1884, gave an address entitled "Memoir on the Loch Katrine Undertaking and its engineers." Mr Warren's reminiscences over such a great span of years gave the meeting an inspiring sense of continuity with a fine tradition. He later presented the awards to the winner of the Charles Hawksley Prize and the three winners of the Miller Prize. A congratulatory telegram was received from the President and read to the Meeting.

The idea of an evening devoted to discussion having proved successful in the past, the first ordinary meeting was devoted to discussion of construction problems and proved to be a well-attended and lively meeting. A very interesting lecture on "The functions of the civil engineer in Courts of Law—observations of a Queen's Counsel" was given by Sir John F. Cameron, D.S.C., Q.C., Dean of the Faculty of Advocates, and attracted a large attendance.

A lecture given to the Association in Inverness by Mr Malcolm M. Morrison (AM) was repeated in Glasgow. He spoke on "Coast protection methods" dealing in particular with Scottish problems and recent works. The Joint Meeting with the Institution of Structural Engineers was addressed by Mr F. A. Partridge (M), who dealt with "The plastic theory as applied to the design of mild steel structures." This led to a lively and, at times, controversial discussion.

As a special feature to mark the seventieth Anniversary of the Association, two lectures were given by Professor E. C. W. A. Geuze, of the University of Delft, who travelled from Holland specially for the occasion and was received by the Lord Provost at the City Chambers. The first lecture dealt with "Foundation problems in the Low Countries" and the second with "Problems concerned with the 1953 floods in the Netherlands." The lectures were memorable, not only for their interest and for the impact of the lecturer's enthusiastic personality, but for the inspiration gained from the example of the Dutch struggle with the sea.

The average attendance at meetings was 74, the maximum being 130 and the minimum 42.

Four meetings were held in Inverness and, as in Glasgow, one of these took the form of a discussion evening. A very interesting lecture by Professor Jack Allen (M), of the University of Aberdeen, on "Hydraulic scale models" with many illustrations from models of well-known works and river estuaries opened the series. The lecture referred to above by Mr Morrison was first given in Inverness. Mr Duncan Kennedy (M) gave a lecture on "Some earlier methods of construction abroad" and aroused great interest by his description of the varied methods encountered in different parts

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* In this Appendix the following abbreviations are used:—Member (M); Associate Member (AM); Graduate (G); Student (S).

of the world before the spread of mechanization. The Inverness meetings have proved their value and considerable enthusiasm is shown at the meetings. The average attendance was 57, a remarkably high figure for such a relatively sparsely populated area.

The Annual Dinner of the Association was held in November, when Mr David M. Watson, President, attended, accompanied by the new Secretary, who was introduced to members and replied to the Chairman's introduction. The toast of the Institution was proposed by Dr T. J. Honeyman, Rector of the University of Glasgow, and replied to by the President. 290 members and guests attended.

The 1954 Summer Meeting of the Association was held at Forest Hills Hotel near Aberfoyle and the Glasgow Water Undertaking was visited.

The Graduates' and Students' Section are proud that one of their members won the Charles Hawksley Prize, and three won Miller Prizes for Papers presented in the previous Session. The response this Session for Graduates' and Students' Papers has been excellent and three evenings have been devoted to them. Mr H. B. Sutherland (AM), Chairman of the Graduates' and Students' Section, has not only continued to create enthusiasm in the Graduates and Students but has worked hard to encourage Corporate Members to support these meetings. The Luncheon meeting was very successful and well-attended, Mr William Linn (M), the Association representative on Council, giving an interesting account of the activities of the Council and its Committees.

The average attendance at Graduate and Student meetings was 39.

The section had an interesting visit to a large steel works at Motherwell.

Mr William Linn (M), Past Chairman, has again accepted nomination as representative of the Association on the Council, and the Committee desire to express their appreciation of the enthusiasm he brought to the task in the past Session.

NORTH-WESTERN ASSOCIATION

Mr D. C. Milne (M), Chief Engineer, Manchester Ship Canal Company, was elected Chairman for the Session. In his Chairman's Address, delivered in Manchester on the 19th October, 1954, Mr Milne gave an account of the construction of "The Queen Elizabeth II Dock," a new oil dock at Eastham, which had been recently completed. Excellent films taken during the construction of the work and also during the model tests added greatly to the interest of the subject, particularly with regard to the verification in the completed dock of the results of investigations by model tests on the behaviour of vessels entering the lock.

Nineteen meetings, including five Joint Meetings with Local Associations of other Institutions and Engineering Societies, were arranged during the Session, eleven at Manchester, four at Liverpool, and one at Preston. The subjects dealt with included: the peace-time activity of the Corps of Royal Engineers; the work of the Road Research Laboratory; applications of reinforced concrete in the gas industry; the construction of the Haweswater Aqueduct; legal aspects of civil engineering contracts; structural problems in the design of buildings for the atomic factory at Capenhurst; aspects of airport development; influence of site conditions on the design of earth dams; retaining wall design.

The Students' Section held a meeting in Manchester at which Graduates and Students gave short talks on their experiences during the first years of professional employment. Many Graduates, Students, and undergraduates took part in the discussion which followed.

A visit was made to Liverpool where the Mersey Docks and Harbour Board are carrying out a large dock reconstruction scheme. A trip was also made along the Manchester Ship Canal from the Manchester Docks to Runcorn where the members were entertained to lunch at Bridgewater House.

In September 1954, a party of 38 Graduates and Students from the North-Western, Yorkshire, Midlands, and London Associations made a nine-day motor-coach tour in Scotland to visit a number of hydro-electric schemes under construction or operative. The weather was kind and a very interesting tour was thoroughly enjoyed, the last day being spent in the English Lake District.

A dance, the first to be arranged by the Association, was held in Manchester on the 21st January, 1955, and as a result of the success of this function it has been decided to make it an annual event.

The President and Secretary of the Institution attended the Annual Dinner at the Midland Hotel, Manchester, on the 10th March, 1955, when there was an attendance of 138 members and guests. Among the principal guests were the Lord Mayor, the Dean of Manchester, and representatives of the University, the College of Technology, and other engineering societies in the north-west.

Professor R. J. Cornish (M) completed three years in office as representative of the Association on the Council of the Institution, and Mr J. E. Harben (M) has been nominated as his successor.

MIDLANDS ASSOCIATION

Mr J. E. Dumbleton (M), Consulting Engineer, took over the Chairmanship of the Association from Mr T. H. P. Veal (AM). In his Address at Birmingham on the 14th October, 1954, and at Loughborough on the 20th October, 1954, he discussed the position of the Local Associations and then dealt with water supply, sewerage and sewage purification from ancient Egypt to the present time.

Fourteen Meetings were held during the Session—seven in Birmingham, four in Loughborough, and two in Nottingham. In addition a Joint Meeting was held in Birmingham with the Institutions of Mechanical and Electrical Engineers. The subjects dealt with were varied and included bridges, concrete, law, aircraft, hydraulics, and nuclear physics.

There were three Graduates' and Students' Meetings which in two cases were held jointly with (*a*) the Institution of Structural Engineers, and (*b*) the Institutions of Mechanical and Electrical Engineers.

A Graduates' and Students' Dance was held jointly with the Institutions of Mechanical and Electrical Engineers.

For the fourth consecutive summer an excursion was arranged which included dinner at Stratford-upon-Avon and a visit to the Shakespeare Memorial Theatre. A visit was first made to the University of Birmingham where tea was taken. About 80 were present including the then President, Mr W. P. Shepherd-Barron.

The Annual Dinner of the Association was held in Birmingham on the 10th November, 1954, and was attended by the President, Mr David M. Watson, and Secretary, the principal guests being the Lord Mayor of Birmingham and Dr Aitken, the Vice-Chancellor of the University of Birmingham.

Mr C. A. Risbridger (M) continued to represent the Association on the Council of the Institution and Sir Herbert Manzoni (M) continued to serve as a member of Council.

NORTHERN COUNTIES ASSOCIATION

Mr T. H. Bryce (M) was installed as Chairman for the Session 1954–55 and delivered his address at Newcastle, Middlesbrough, and Carlisle on "The history of bridges."

Eight meetings have been held at Newcastle, the subjects of the Papers being :— the Owen Falls hydro-electric scheme ; Nevilles Cross Bridge ; problems of road traffic control ; siting and design of harbours ; the design and construction of No. 8 dry dock, North Shields ; the reconstruction of the fish quay at North Shields ; the fundamental properties of matter.

Six meetings were held at Middlesbrough. The subjects discussed included road traffic problems in north-eastern England ; the evolution and development of water pipes ; some practical applications of soil mechanics ; tests on road aggregates ; surface development in a mining area.

Two meetings have been held at Carlisle. The Chairman gave his Address and Professor W. F. Cassie lectured on Continental motor-ways.

Mr Alastair Storrar (AM) was elected Chairman of the Graduates' and Students' Section and seven meetings have been held in Newcastle. The subjects discussed were public health engineering in Northern Rhodesia ; the evolution and development of water pipes ; heavy-duty compressed-air plant ; local government engineering ; and civil engineering in India.

There has been an increased attendance at meetings during the session.

In January a Dinner and Dance was held in Durham Castle. Members and their ladies had a pleasant evening in spite of the wintry conditions and hazardous travelling.

The Annual Dinner was held at the Royal Station Hotel, Newcastle upon Tyne, and, in addition to Members and their guests, was attended by the President and Secretary of the Institution, the Lord Mayor of Newcastle upon Tyne, the President of the Consular Corps, representatives of the Port Authority, kindred engineering institutions, and other distinguished persons. Sir Anthony Eden, P.C., M.P., Foreign Secretary, was unfortunately unable to be present but sent a message of good-will to the gathering.

Last July Members and their Ladies visited Durham, where they were conducted by Colonel C. F. Battiscombe, O.B.E., M.A., on a tour of the Monks' Dormitory and other places of interest in the Cathedral. After lunch the party travelled up Weardale to Burnhope Reservoir and inspected a new filtration plant. Leaving Burnhope after tea, kindly provided by Sunderland and South Shields Water Company, the return journey was made via Barnard Castle calling at the Bowes Museum. The party was conducted on a tour of the Fine Arts and Historical Sections by Dr Thomas Wake, Curator of the Museum.

In September there was a visit to the Wilton Works of Messrs Imperial Chemical Industries, and the Tyne Improvement Commission kindly made their launch available for Members and their Ladies to take a trip on the River Tyne and to inspect some of the river works.

YORKSHIRE ASSOCIATION

The 1954–55 Session opened on the 24th September, 1954, when the new Chairman, Mr J. G. Taylor (M), Divisional Road Engineer, North-East Division, Ministry of Transport and Civil Aviation, as successor to Mr D. Currie (M), delivered his Address at a meeting held in Sheffield. The Address was repeated at Hull on the 13th October. Mr Taylor very briefly covered the history of roads in this country and in more detail referred to road works carried out in the inter-war years. Reference was made to the post-war trends in highway planning and in particular the demand for and design of "motor roads."

During the Session a total of eight Ordinary Association meetings have been held, one meeting in Scunthorpe, one in York, three in Leeds, and three in Sheffield. The meeting at Scunthorpe was held jointly with the local branch of the Lincolnshire Iron and Steel Institute when two Papers dealing with the design and operating of handling and treatment systems for liquid radioactive wastes were presented. In addition one meeting was held jointly with the Yorkshire Branch of the Institution of Structural Engineers and one jointly with the North Midland Centre of the Institution of Electrical Engineers.

Subjects covered by Papers were hydro-electric development in Uganda, design of high buildings, prestressed concrete in short-span highway bridges, coast protection, geophysical surveying, and hydraulic scale model investigations.

The average attendance at Association meetings was about 50.

Three meetings of the Graduates' and Students' Section have been held in addition to the Annual General Meeting. Papers on hydraulic valve design, tests on steel frame buildings, model tests on seepage through dams, railway underbridge construction, and permanent-way layouts have been presented. In two cases Papers were preceded by visits to Messrs J. Blakeborough & Sons and to a store shed and track layout recently carried out by the North-Eastern Region of British Railways. A successful dance organized jointly by the Graduates' and Students' Sections of the Institutions of Civil, Mechanical, and Electrical Engineers was held on the 17th December.

The Hull and East Riding Branch of the Association held six meetings including the Chairman's Address. Papers covered dock construction, flood control, structural design, and trade effluent problems. One meeting was devoted to Papers presented by Graduates and Students. A visit was arranged jointly with the Yorkshire Branch of the Institution of Structural Engineers to the new factory of the Imperial Typewriter Company at Hull. Branch meetings continue to receive good support from members; the average attendance being about 50.

The 34th Annual Dinner of the Association was held on the 23rd March, 1955, at the Grand Hotel, Sheffield, and was attended by 163 guests, members, and their friends. The President and the Secretary of the Institution were present and among the guests were the Lord Mayor, the Vice-Chancellor of Sheffield University, and the Presidents and Chairmen of kindred associations and their ladies. During the Dinner the Chairman and Vice-Chancellor referred to the recent appointment of Mr N. S. Boulton (M), Past-Chairman of the Association, to the newly created Chair of Civil Engineering at Sheffield University.

SOUTH-WESTERN ASSOCIATION

At the beginning of the Session Mr B. E. Ireland (M) handed over the office of Chairman to Mr N. A. Matheson (M), who is Engineer-in-Chief of the Port of Bristol Authority.

The Honorary Secretaryship also changed from Mr G. C. Mander (M) to Mr John R. Freeman (AM). At the opening meeting Mr Ireland made a presentation to Mr Mander with the thanks of the Association for his three years as Honorary Secretary. The Honorary Assistant Secretaryship changed too, from Mr R. J. Bradbury (G) to Mr J. B. Shaw (G) for a short while ; and then to Mr R. J. Collins (G). In Plymouth Mr J. R. Stanley (AM) handed over Local Association matters to Mr W. P. Hawkyard (AM). The Association continued to be represented on the Council of the Institution by Mr G. E. Scott (M).

A total of fifteen ordinary meetings and the usual Annual General Meeting, were held during the year ; of these nine were held in Bristol, five in Plymouth, and one in Exeter ; they included four joint meetings with the Institution of Structural Engineers and one Graduates' and Students' evening in Bristol.

At the opening meeting Mr N. A. Matheson (M) gave his Chairman's address on sea-going docks from the civil engineering viewpoint. Lectures at the other meetings included city planning and reconstruction, model experiments on the Thames Estuary, dock construction, overcoming frost-heave in a cold store, the design of a nuclear-reactor building and of a large British prestressed concrete bridge, and the investigations and remedial measures into a large-scale landslip.

Two day visits were made ; one to the new London Airport at Heathrow to see the new construction ; and one to Gloucester to see a new sewage works, the new Maisemore Bridge construction, and the work being carried out to the leaded roof of Gloucester Cathedral. A half-day visit was arranged to Portishead " B " Power Station. Each meeting was attended by about 35 members.

A Graduate member, Mr G. E. Peart (now AM), won a Miller Prize for his Paper on " Railway track maintenance " and the presentation was made at the beginning of the Session. Mr F. K. Smith (S) of Bristol Authority, presented a Paper to the Association on " Designs for decking Clifton Suspension Bridge." In December, the Students and Graduates joined with the Institutions of Mechanical and Structural Engineers and the Royal Aeronautical Society to hold a successful Christmas Ball.

The Annual Dinner and Dance of the Association was brought forward from March to December this Session and 100 members and their guests attended to meet the President of the Institution, Mr David M. Watson, and the Secretary, Mr Alexander McDonald. The evening was greatly enjoyed by all present.

For the fourth year in succession monthly luncheons have been held in Bristol and have become a successful part of the Association's activities. Twenty-five to thirty members and their guests usually attend and the meal is followed by a short talk on some topical engineering matter.

NORTHERN IRELAND ASSOCIATION

Mr E. A. F. Johnston (M) succeeded Mr J. J. Hartley (AM) as Chairman for the Session and delivered his address on the 18th October, 1954, dealing mainly with engineering management.

Seven ordinary meetings were held, two of them being joint meetings with the Northern Ireland Association of the Institution of Electrical Engineers. As is the custom, one meeting was organized by the Graduates and Students. Apart from the Chairman's address, one Paper was by an Associate Member and one by a Graduate. The remaining lectures were given by visitors to Belfast.

During the Session two visits have been arranged. A welcome feature has been the introduction of visits for Junior Members of which there have been three to date ; two others are proposed.

The President and Secretary of the Institution visited Belfast for the Annual Dinner of the Association held in the Midland Hotel on the 10th February, when about

93 members and guests were present. The principal guests were the Minister of Finance and the Lord Mayor.

A successful social occasion in the form of a supper dance was held on the 22nd April, 1955.

Attendances at meetings of the Association have continued to be satisfactory. The present strength of the Association is :—33 Members, 194 Associate Members, 86 Graduates, 88 Students.

SOUTHERN ASSOCIATION

On the 14th October, 1954, Mr J. W. Hunter (M), Civil Engineer Manager, H.M. Dockyard, Portsmouth, succeeded Mr F. W. Taylor (AM) as Chairman. His address, delivered at Portsmouth, was entitled "Civil engineering welding and cutting" and dealt with problems and works in this field of engineering with which he had been concerned during his career with the Admiralty. It was illustrated with lantern slides and a large number of interesting exhibits.

During the Session ten meetings have been held—two at Portsmouth, two at Brighton, three at Chichester, two at Southampton, and one at Bournemouth. They covered the following subjects : plastic design methods for framed structures, water supply, modern rock tunnelling methods and equipment, the requirements of a civil airport, surveying from air photographs, the design of nuclear-reactor buildings, and the design and construction of the Owen Falls, Uganda, Hydro-Electric Scheme. The Vernon Harcourt Lecture, "Siting and design of harbours," was repeated by Mr E. J. Buckton (M) at Southampton on the 14th April, 1955, and Mr R. H. Coates (AM) delivered a Paper on "The unpredictable nature of civil engineering contracts" after the Annual General Meeting. Of the ten meetings held, one was joint with the Southern Centre of the Institution of Electrical Engineers, one with the Southern Branch of the Institution of Mechanical Engineers, and one with the South Eastern District of the Institution of Municipal Engineers, members of the Institution of Structural Engineers being invited to attend the latter. Attendance at meetings has continued to be high, being an average of 85 for the Session.

In addition to the Session's ordinary meetings, the Graduates' and Students' Section has been particularly active, having held five meetings in Chichester, Brighton, and Portsmouth when attendance has been in the neighbourhood of 70 members. The section has also organized three visits to take place during the summer months.

The Association this year departed from its more recent custom of holding an Annual Summer Meeting and held instead an Annual Dinner and Dance. This took place on the 13th May, 1955, fairly late in the year, to enable members and guests to obtain the maximum benefit from the fact that it was held at the Royal Norfolk Hotel, Bognor Regis. The President (Mr David M. Watson) and the Secretary attended, and guests included the Commander-in-Chief, Portsmouth, Admiral Sir George Creasy, the Civil Engineer-in-Chief, Admiralty, and representatives from kindred associations.

A number of visits were organized during 1954 and were well attended. These included the Ministry of Supply Experimental Establishment at Christchurch, the R.E. Transportation Centre at Longmoor, and the Hampton Court works of the Metropolitan Water Board.

Mr P. E. Sleight (M) continued to represent the Association on the Council for the Session.

SOUTH WALES AND MONMOUTHSHIRE ASSOCIATION

Mr F. V. M. Bell (M), General Superintendent Construction of the National Oil Refineries, Llandarcy, was elected Chairman of the Association and took office on the 4th October, 1954. The Chairman's address entitled "The civil engineers' Charter obligations" evoked a lively interest among members.

Six ordinary meetings have been held in Cardiff, one in Margam, two in Swansea, and one in Carmarthen. The average attendance for the Session was 60. The meeting which attracted the largest attendance (84) entitled "Simple plastic theory" was given by Dr B. G. Neal (AM) at Cardiff. One joint meeting was held in Margam with the Institution of Municipal Engineers, and one with the Institution of Structural Engineers in Cardiff. Visits were organized to Abbey Works, Margam Velindre, and Llandarcy. During the Session, one ordinary meeting and one visit took place in the Students' and Graduates' Section.

The Fourth Annual Ball proved to be as enjoyable as the three previous ones.

The Annual Dinner held at the Royal Hotel, Cardiff, on the 16th March, 1955, was again a great success. The President, Mr David M. Watson, was present, and the Association was also pleased to welcome the new Secretary, Mr Alexander McDonald, on his first visit to Wales since his appointment.

Lt-Col. Arthur Borlase (M) was elected Council Member in succession to Lt-Col. R. H. Edwards (M), who retired after having served for five years.

The Committee has met before each ordinary meeting and the average attendance has been 14. Mr E. C. Roberts (M) represented the Association on the Ministry of Works Regional Advisory Committee.

The Honorary Secretary, Mr T. Leslie Lowe (M), has attended four meetings in London of the Honorary Secretaries of Local Associations.

EDINBURGH AND EAST OF SCOTLAND ASSOCIATION

The Chairman for the Session has been Mr Leslie H. Dickerson (M), Chief Assistant Hydraulic and Civil Engineer, North of Scotland Hydro-Electric Board, who succeeded Mr Colin F. Armstrong (M), Superintending Civil Engineer, H.M. Dockyard, Rosyth. Mr Dickerson in his inaugural address on the 13th October, 1954, traced the developments which had taken place in recent years in the technique of rock tunnelling. Dealing briefly first with the earlier Scottish Schemes, such as the Lochaber tunnel, the Grampian Scheme tunnels, and the Galloway Scheme, he then described some of the tunnels, amounting in aggregate to 127 miles, driven during the ten years' existence of the Hydro-Electric Board. With so much work of this nature there has grown up a force of really skilled tunnel men which partly accounted for the remarkable improvement in output in recent years. Other features were : (1) in drilling, the pneumatic piston percussion drifter had been greatly improved and the introduction of the light air-leg equipment and special drill steel and tungsten carbide-tipper bits had marked an important advance ; (2) in firing, the modification of the pattern of holes drilled and the order of firing had helped to improve the scatter and size of dislodged materials, for mucking is apt to be the governing factor in the attempt to pull one complete round per shift ; (3) in mucking, the hand shovel has been replaced by some form of mechanical loader, the tunnel being made somewhat larger in some cases in order to accommodate the gear.

Eleven meetings were held in Session 1954–55, not including the Annual Dinner and the Joint Dance with the Scottish Section of the Institution of Electrical Engineers, but including the inaugural meeting, two special Graduates' and Students' meetings, two outside meetings at Aberdeen and Perth respectively, and the Annual General Meeting. The subjects dealt with were "Administration of contracts" (for Students), "Owen Falls, Uganda, Hydro-Electric Development" (Joint Meeting with I.E.E.), "Civil engineering aspects of the Forth road bridge," "The main drainage of the City of Edinburgh," "The American approach to civil engineering problems" (for Students), "The Queen Elizabeth II Dock, Eastham" (at Aberdeen), "Mining developments in Scotland," and "Quality control for mass concrete." At the meeting in February 1955, one Graduate Member and two Student Members read Papers which were registered for the 1954–55 Miller Prize Competition. Films were shown after the Annual General Meeting.

The average attendance at the Ordinary Meetings was about 100 as compared with about 80 in 1953–54. The record number of 169 were present at the Joint Meeting for Sir Charles Westlake's Paper. The attendances at Aberdeen and Perth and at the two Students' Meetings were satisfactory though smaller.

Three Papers, read on the 10th February, 1954, were registered for the 1953–54 Miller Prize Competition and all three authors were awarded prizes, viz. :—Messrs Addly (G), Drummond (S), and Harvey (G). On the 21st October, 1954, Mr Harvey was also awarded the 1954 Institution Medal and Premium (Local Associations) for his Paper "Concrete work on Upper Glendevon Reservoir."

The Annual Dinner was held in November 1954 when the President, Mr David M. Watson, and the Secretary, Mr Alexander McDonald (M), were welcomed by their fellow-Scots, the record number of 139 being present.

Recent visits to engineering works included—Colvilles' Clydebridge steelworks ; Caledonia flour mill (Leith Docks) ; Broadside water filtration plant, Stirlingshire ;

Glen Shira hydro-electric scheme ; Edinburgh western boundaries sewer, and Easthouses colliery.

One Christmas Lecture for secondary-school pupils was arranged by the Joint Committee on Lectures to Schools under the auspices of the Institution of Mechanical Engineers. Sir William Wallace entitled his Lecture "Ship Stabilizers."

As a result of an offer by Principal Whittaker (M), members of the Association have now the use of the College Library of the Dundee Technical College as well as that of the Heriot-Watt College, Edinburgh.

Mr W. P. Haldane (M) continues to represent the Association on the Scottish Advisory Committee for Codes of Practice and Mr S. A. Findlay (M) on the Joint Advisory Panel on Building of the Ministry of Works.

Two members of Council are now members of the Committee of the Edinburgh and East of Scotland Association, viz., Mr J. L. White (M), the Association's representative, and Mr A. A. Fulton (M), who was elected by the Annual Ballot in 1954.

APPENDIX III

DEATHS AND RESIGNATIONS

The full list of deaths is as follows (E. refers to election, T. to transfer, and A. to admission) :—

Members (61).—Robert Baker, *O.B.E.* (E. 1915) ; Robert Glover Baxter, *O.B.E.*, M.A. (E. 1927, T. 1946) ; *Sir* Ernest Albert Seymour Bell, *C.I.E.*, F.C.H. (E. 1895, T. 1913) (*former Member of Council*) ; Thomas Bell, *E.D.* (E. 1927, T. 1946) ; Ernest Alexander Black (E. 1915, T. 1941) ; Robert Herbert Blackburn (E. 1909, T. 1945) ; Hubert Cecil Booth (E. 1897, T. 1910) ; Alan Boothman (E. 1929, T. 1943) ; Ernest Andrew Borg (E. 1935) ; Henry Farre Bowen (E. 1906, T. 1920) ; Robert James Boyd, M.E. (E. 1907, T. 1922) ; George Bransby-Williams (E. 1897, T. 1909) ; Robert Bruce (E. 1904, T. 1926) ; Charles Augustus Carlow (E. 1938) ; Herbert Chatley, D.Sc.(Eng.) (E. 1920, T. 1928) ; Edmund Graham Clark, *C.B.E.*, *M.C.*, M.Sc. (E. 1914, T. 1938) ; William Cecil Clemens, *O.B.E.* (E. 1912, T. 1938) ; Walter Ponsonby Shaw Cockle, B.Sc.(Eng.) (E. 1928, T. 1945) ; George Edwin Bohun Coulcher (E. 1910, T. 1926) ; Percy John Cowan (E. 1903, T. 1927) ; *Col.* Cyril Murton Croft, *D.L.*, *J.P.* (E. 1935) ; *Sir* Maurice Edward Denny, *Bart.*, *K.B.E.*, B.Sc. (E. 1918, T. 1924) ; Joseph Henry Dobson, *D.S.O.*, D.Eng., D.Sc. (E. 1904, T. 1919) ; John Samuel Duncan, B.Sc. (E. 1936, T. 1945) ; Francis Maurice Gustavus Du-Plat-Taylor (E. 1919) ; Somers Howe Ellis (E. 1896, T. 1907) ; John Leonard Eve (E. 1938) ; John Thomas Dungate Fillingham (E. 1911, T. 1931) ; Robert Charles Frain (E. 1920) ; George Fraser (E. 1913, T. 1929) ; Harold Moffat Swire Haughton, B.Sc. (E. 1910, T. 1933) ; David Henderson (E. 1910, T. 1933) ; Herbert Ross Hooper, *O.B.E.*, M.A. (E. 1890, T. 1903) ; Arthur Cyril Jennings, *O.B.E.* (E. 1914, T. 1942) ; Frank Harding Jones (E. 1909) ; *Sir* John McFarlane Kennedy, *O.B.E.*, B.A. (E. 1905, T. 1914) ; Charles Edward Larard, D.Sc. (Eng.) (E. 1895, T. 1913) ; Lawrence Lee-Dunham (E. 1925, T. 1942) ; Albert Edward Leek, *M.B.E.* (E. 1913, T. 1938) ; Percival Arthur Rupert Leith (E. 1932) ; George McIldowie (E. 1913, T. 1936) ; Edward Langlois Montagnon, Ph.D., B.Sc.(Eng.) (E. 1919, T. 1938) ; Alexander Murray (E. 1923, T. 1948) ; Ethelbert Monk Newell, *D.S.O.* (E. 1918, T. 1929) ; Henry Nimmo, *C.B.E.* (E. 1936) ; John Amphlett Parker, *M.C.*, B.Sc.(Eng.) (E. 1921, T. 1932) ; Leslie Preston Parker, *O.B.E.*, B.Sc.(Eng.) (E. 1918, T. 1946) ; James M'Farquhar Petters (E. 1900, T. 1910) ; Frederick Gerald Rappoport (E. 1914, T. 1925) ; Francis Bartlett Richards (E. 1934) ; Leopold Penrose Ridgway (E. 1912, T. 1927) ; William Duncan Lund Roberts (E. 1907, T. 1927) ; Alexander Gibson Smith (E. 1910, T. 1925) ; Harry Abraham Smith, B.Sc. (E. 1920, T. 1938) ; Kenneth Maurice Steven, B.Sc. (E. 1931, T. 1948) ; Theodore Stevens, B.Met., E.M. (E. 1904, T. 1910) ; John Charles Telford, *O.B.E.* (E. 1910, T. 1922) ; John Terrace (E. 1924) ; *Sir* Arthur Watson, *C.B.E.*, LL.D. (E. 1899, T. 1911) ; Rees John Williams (E. 1927, T. 1951) ; Smelter Joseph Young (E. 1910).

Associate Members (66).—Joseph George Alexander (E. 1904) ; Eric Gaylmer Lytton Anderson, B.A., B.A.I. (E. 1929) ; Edward Andrewes, B.Sc. (E. 1905) ; Hugh Ritchie Barr (E. 1905) ; George Maurice Beaumont (E. 1944) ; Roland Hirst Beaumont (E. 1906) ; Thomas Henry Beckett (E. 1928) ; Soden William Biden (E. 1908) ; Frederick Francis Percival Bisacre, *O.B.E.*, M.A., B.Sc. (E. 1914) ; William Charles

Neville Bosworth (E. 1929); William Butterworth (E. 1926); Leonard Onslow Cope (E. 1925); Patrick Cullinane, B.E. (E. 1949); Dennis Cuttiford (E. 1945); Clement Frederick Davis, B.A. (E. 1897); Edward Mervyn Dench, B.E. (E. 1951); Mathrage Romiel Dharmasiriwardhane (E. 1932); Ronald Murray Drynan (E. 1925); Percy Richard Duncan, M.A., B.Sc. (E. 1938); William Ainslie Foulis (E. 1901); Charles Garrod, B.Sc. (E. 1904); John Maxwell Granger (E. 1906); Thomas Daniel Hamilton, *M.C.* (E. 1914); Edward Townsend Hippisley, M.A. (E. 1941); Francis Chaplin Holland, B.Sc. (E. 1920); Leslie Mark Howard (E. 1928); John Hunter (E. 1923); Arthur Daniel Jaffe, *O.B.E.*, M.A. (E. 1905); Arthur Henry Johnstone, B.A.I. (E. 1898); Frederick William Jones (E. 1907); Robert Owen Jones (E. 1912); Jan Kacirek, B.Sc. (Eng.) (E. 1954); Alexander Hamilton Knox (E. 1921); John Crabbe Lambert (E. 1951); Albert Edward Charles Leppard, B.Sc.(Eng.) (E. 1922); William Stevenson MacCulloch (E. 1908); Elwyn Laurence MacLeod (E. 1946); William Forster McMillan, B.Sc.(Eng.) (E. 1922); Arthur George Mansell (E. 1926); Arthur Neville Marchant, B.Sc. (E. 1934); James Herbert Mason, M.A. (E. 1914); Walter Dow Mathieson (E. 1923); John Grove Stanley Middleton, B.Sc. (E. 1916); Ernst Julius Oelsner, D.Eng. (E. 1949); William Morris Prout (E. 1896); Maxwell Robert Puddy, B.Sc. (E. 1950); Leonard Watson Pye (E. 1915); Harvey Robert Sayer (E. 1917); Walter McNeil Shimmin (E. 1920); Sidney Gordon Basil Skingle, *M.B.E.* (E. 1938); Frank James Smith, *M.B.E.* (E. 1934); Robert Spence (E. 1927); Eric Owen Stubbings, *M.C.*, B.Sc.(Eng.) (E. 1915); Percival Wilberforce Turner (E. 1906); Robert Henry Rawlinson Walker, M.A. (E. 1939); Fred Curry Walls (E. 1929); George Waters, B.Sc.(Eng.) (E. 1937); Herbert Nicoll Weldon (E. 1890); Arthur Trevor Yeo West (E. 1946); John Everard White (E. 1938); Stanley White, B.Sc. (E. 1940); Percy Daniel Whitestone, M.A. (E. 1949); *Sir* Herbert Geraint Williams, *Bart.*, *M.P.*, M.Sc., M.Eng. (E. 1910); Thomas Allen Womersley (E. 1915); Charles Edward Woodger, B.Sc. (Eng.) (E. 1928); William Percy Wordsworth, B.Sc. (E. 1917).

Associates (2).—William Hanneford-Smith, F.R.S.E. (E. 1927); Edward Fulton Law (E. 1936).

Graduates (2).—Edward Shepherd Darling, B.A. (A. 1952); Peter Robinson (A. 1954).

Students (4).—Henry Findlay Baird (A. 1952); John Michael Harrison (A. 1952); John Gavin Johnston (A. 1947); Peter Edward Woodman, B.Sc.(Eng.) (A. 1947).

The following resignations have been received:—

Members (16).—*Professor* Charles Andreae (E. 1936); Ewart Sigmund Andrews, B.Sc.(Eng.) (E. 1923, T. 1927); Charles Robert Barlow (E. 1908, T. 1921); Wills Burleigh Close, B.A., B.A.I. (E. 1916, T. 1928); Reginald Alfred Collett, *O.B.E.* (E. 1913, T. 1933); Charles Greenwood, *O.B.E.* (E. 1915, T. 1936); Marcar Sheridan Gregory, *O.B.E.*, *M.C.*, *V.D.* (E. 1914, T. 1928); Frederick Heap Hutchinson, *C.I.E.*, B.Sc.(Eng.) (E. 1947, T. 1948); Richard Daniel Thomas Jones (E. 1913, T. 1922); Walter John Smith Key (E. 1927, T. 1947); Charles Clement Paul, *O.B.E.* (E. 1915, T. 1930); George Ewart Rhodes, *C.B.E.* (E. 1913, T. 1926); Samiullah Shah, B.Sc. (E. 1922, T. 1937); Richard Alan Stuart Thwaites, B.Sc.(Eng.) (E. 1913, T. 1928); Joseph Stanley Westerdale, B.Sc. (Eng.) (E. 1921); Ernest Wilson (E. 1909, T. 1933).

Associate Members (54).—Herbert Arthur Dyke Acland, B.A. (E. 1912); Alfred Thompson Alkin (E. 1937); John Duncan Allan (E. 1922); George Robert Anderson, B.Sc.(Eng.) (E. 1917); Motumal Asudamel Asrani, B.Sc. (E. 1920); Horace William Ash, B.Sc.(Eng.) (E. 1919); Edgar Maximilian Baerlein, B.A. (E. 1908); Henry Cooke Bell (E. 1904); Owen William Berry (E. 1925); Arnold Edmund Harker Bousfield, B.A., B.A.I. (E. 1932); Joseph Bradley, *O.B.E.*, B.A.I. (E. 1930); Archibald Bryce Bremner (E. 1929); Henry John Bretton (E. 1919); Horace Roy Brown, B.Sc.(Eng.) (E. 1944); Frank Bullivant (E. 1904); George William James Clark, *M.B.E.*, B.Sc. (Eng.) (E. 1919); Edwin Hugh Colgrove (E. 1919); Edgar Hugh Collcutt (E. 1912); George Smith Collie (E. 1930); Eric William David Davies, B.Sc. (E. 1919); John Anthony Derry, B.Sc. (E. 1924); James Dyke (E. 1936); Douglas Gordon French, B.A., B.Sc. (E. 1910); Reginald Glanfield (E. 1910); John Kilgour Grant (E. 1920); Herbert Willan Hayton, M.Eng. (E. 1923); Allan Peter Hitchens, B.Eng. (E. 1919); William Hodgson, B.Sc.(Eng.) (E. 1918); Gerald William Hodsman (E. 1915); Donald Henry Holley, *M.C.* (E. 1919); James Alexander Houston, B.A., B.E. (E. 1928); Charles Langbein (E. 1925); Guy Talbot Lemon, *O.B.E.*, B.A. (E. 1912); Frederick Thomas Littlejohn (E. 1916); Eric McGregor, *M.C.* (E. 1917); Alexander Davis

Mackay, B.Sc. (E. 1926); Frederick Edward Meyer (E. 1929); *Sir* Christopher Norman Musgrave, *Bart.*, B.A., B.A.I. (E. 1924); Thomas Martin Naylor, M.Sc. (E. 1919); George Milner Pickard (E. 1930); Harold Llewellyn Quick, B.Sc.(Eng.) (E. 1921); Wilfrid John Robinson, B.Sc.(Eng.) (E. 1932); Robert Henry Price Ronayne (E. 1915); Howard Lamont Smith (E. 1929); Alick Lionel Tackley, B.Sc. (E. 1919); Edward Francis Lawrence Taylor, B.A. (E. 1918); Henrik Thackeray (E. 1910); William George Tomlinson, B.A., B.A.I. (E. 1920); Arthur Cecil Tregoning, M.C.E. (E. 1936); John Frederick Wallace, B.Sc. (E. 1932); John Gutteridge Ward (E. 1917); Gordon Welch, B.Sc.(Eng.) (E. 1926); Kenneth Ewart Westwood, B.Sc. (E. 1946); Alexander William Wilson, *M.C.* (E. 1924).

Graduates (11).—John Cyril Briggs, B.Sc. (A. 1952); Neville Teare Deakin, B.Sc. Tech. (A. 1952); Michael Garrod Elston, B.Sc.(Eng.) (A. 1953); Jan Troye Faye, B.Sc. (A. 1953); John Oliver Hulbert (A. 1952); Kumpati Balasubrah Manyam, M.Sc.Tech., B.E. (A. 1952); Frederick Harold Needham, B.Sc.(Eng.) (A. 1952); Alan Ernest Peterson, B.Sc. (A. 1953); James Archibald Gordon Sneddon (A. 1952); Guy Valentine Tennant (A. 1953); James Crawford Thompson, B.Sc. (A. 1953).

Students (41).—John Michael Alty (A. 1951); George Graham Anthony (A. 1952); Gerald Christopher Awre (A. 1949); Bernard Frederick Barrett (A. 1948); Bryant Godfrey Bayliffe (A. 1951); George Alexander Bell (A. 1949); Eric Dunstan Bill (A. 1952); Donald Ross Bishop (A. 1947); William Rupert Blain (A. 1952); Colin Stanley Brotherton (A. 1950); James Joseph Bunting (A. 1951); Stephen William Henry Butler (A. 1951); Alan Frederick Christmas (A. 1948); Peter Clarkson (A. 1950); Alexander Coull (A. 1953); Philip John Davies (A. 1951); David Gerald Fairhurst (A. 1950); William John Charles Ford (A. 1948); Robert Guy Greswold (A. 1947); Michael Brian Habgood (A. 1949); Albert Harper, B.Sc. (A. 1948); Roger Mansell Hollins (A. 1953); Roy Stuart Kermeth (A. 1951); Peter John Ford Lane (A. 1953); Austin Laybourne (A. 1949); Graham Trevor Ling (A. 1947); David McDiarmid (A. 1953); James Sugden Marshall (A. 1952); Keith Michael Newton (A. 1950); Philip Alan Northfield (A. 1952); Roy Ernest Norfolk (A. 1950); John Laidlaw Phimister (A. 1949); Hugh Arthur Ellwood Pitkin (A. 1948); Rowland Francis Pocock (A. 1951); Richard Vernon Reilly (A. 1949); Allan William Saunders (A. 1949); David George Shannon (A. 1949); Stanley Gordon Stock (A. 1951); Kanapathippillai Thirugnana Sundaram (A. 1950); David Noel Vincent (A. 1948); David Zucker (A. 1948).

RESERVE FOR REPAIRS AND RENEWALS TO STRUCTURE, FURNITURE, FITTINGS AND MACHINERY

–54		£	s.	d.	£	s.	d.
83	Balance, per last account				3,745	17	0
	Add Credits during the year—						
50	Allocation of Interest on Investments				250	0	0
	Institution Revenue Account—						
00	Amount provided for the year, *page 630*				4,000	0	0
33					7,995	17	0
87	Less Net Expenditure during the year				5,532	7	5
46	Balance carried forward, per Balance Sheet, *page 628*				2,463	9	7

PUBLICATIONS EXPENDITURE

		£	s.	d.	£	s.	d.
	Expenditure during the year—						
131	Proceedings, etc.				26,411	15	11
936	"Chartered Civil Engineer"				2,147	19	1
71	"Géotechnique"—Expenditure less Receipts				—	—	—
-	"General Conditions of Contract"				1,232	1	8
149	Charters, By-laws and Lists of Members				52	17	9
796	Engineering Abstracts				731	17	2
729	Salaries and Pension Premiums				7,377	19	6
430	Reporting and Sundries				484	11	8
542					38,439	2	9
	Deduct:—						
	Credits for Advertisements,						
62	Sales, Subscriptions, etc.	20,768	16	5			
-	"Géotechnique"—Receipts less Expenses	212	12	2			
					20,981	8	7
80	As per General Revenue Account, *page 630*				17,457	14	2

RESEARCH

		£	s.	d.	£	s.	d.
94	Sales of Proceedings and Reports				2,789	16	7
	Less Expenditure during the year—						
16	Salaries and Pension Premiums	1,479	13	4			
28	Travelling to Committees	32	5	9			
06	Printing of Sundry Reports	596	13	5			
3	Sundry Expenses	58	16	10			
53					2,167	9	4
it 59	As per General Revenue Account, *page 630*				622	7	3

THE INSTITUTIO

BALANCE SHEE

31 March 1954 £					£	s.	d.	£	s.
	INSTITUTION CAPITAL ACCOUNT								
414,759	*As per last Account*							414,758	16
	INSTITUTION BUILDING SINKING FUND								
	As per last Account				2,446	9	1		
	Add : Allocation for year to date				450	0	0		
	Interest on Investments and Income Tax refunded				88	9	9		
2,446								2,984	18
	REPAIRS AND RENEWALS RESERVE								
3,746	*As detailed on page 627*							2,463	9
	GENERAL REVENUE ACCOUNT SURPLUS								
	Balance at 31st March, 1954				26,896	9	0		
	Add : Surplus for the year to date, *page 630* . .				8,206	3	3		
26,896								35,102	12
447,847								455,309	17
	CREDITORS—								
17,604	Sundry Creditors							21,832	15
	REVENUE IN SUSPENSE								
49,255	Proportion of subscriptions, examination fees, etc. received relating to the period after 31st March, 1955							46,369	3
514,706								523,511	16
	TRUST FUNDS								
	Capital Accounts				69,604	9	0		
	Unexpended Income—								
	Balance per last account . .	8,441	1	8					
	Add : Income received for the year to date	2,246	17	5					
		10,687	19	1					
	Less : Expenditure on Scholarships, Prizes, Lectures, etc. .	1,428	6	1					
					9,259	13	0		
77,971								78,864	2
	SEA ACTION COMMITTEE ACCOUNT—								
	Balance at 31st March, 1954				990	11	0		
	Add : Interest and other receipts for the year to date				41	8	1		
991								1,031	19
593,668								£603,407	17

REPORT OF THE AUDITORS TO THE ME

We have obtained all the information and explanations which to the best of our knowle kept by the Institution so far as appears from our examination of those books. We have e of account. In our opinions and to the best of our information and according to the expl 31st March, 1955, and the General Revenue Account gives a true and fair view of the surplu

London, 13th May, 1955

L ENGINEERS

1st MARCH, 1955

1 March 1954 £			£ s. d.	£ s. d.
	FREEHOLD PROPERTY—Institution Building—			
75,768	*At cost, as per last Account*		..	375,767 16 10
	INSTITUTION INVESTMENTS at or			
85,878	under cost		..	85,878 7 5
	Market value, £78,543—(1954, £78,939)			
	INSTITUTION BUILDING SINKING FUND			
2,376	Investments at cost		2,896 9 1	
	Market Value, £2,687—(1954, £2,365)			
70	Cash awaiting investment		88 9 9	
				2,984 18 10
7,017	**DEBTORS**			4,365 0 4
43,597	**BALANCES AT BANKERS AND CASH IN HAND**			54,515 13 0
14,706				523,511 16 5
	TRUST FUND ASSETS—			
	Capital :—			
	Investments	69,494 10 6		
	Market value, £57,376—(1954, £58,546)			
	Balance at Bankers	109 18 6		
			69,604 9 0	
	Unexpended Income—			
	Investments	437 0 11		
	Market value, £352—(1954, £360)			
	Balance at Bankers	8,822 12 1		
			9,259 13 0	
77,971				78,864 2 0
	SEA ACTION COMMITTEE ACCOUNT—			
	£700 3% Savings Bonds 1955/65 at cost . . .		700 0 0	
	Market value, £668—(1954, £679)			
	Balance at Bankers		331 19 1	
991				1,031 19 1
3,668				£603,407 17 6

A. McDONALD, *Secretary*

E INSTITUTION OF CIVIL ENGINEERS.

ere necessary for the purposes of our audit. In our opinions proper books of account have been ve Balance Sheet and annexed General Revenue Account which are in agreement with the books the said Balance Sheet gives a true and fair view of the state of the Institution's affairs at the ded on that date.

A. RAE SMITH (Deloitte, Plender, Griffiths & Co.) } Auditors.
J. A. JOHNSTON

GENERAL REVENUE ACCOUNT FOR THE YEAR ENDED 31st MARCH, 1955

INCOME.

1953–54 £		£	s.	d.	£	s.	d.	£
70,619	Subscriptions received applicable to the financial year 1954–55					..		71,469
9,306	Entrance Fees					..		10,035
250	Life Composition					..		—
	Interest, Dividends, etc.—							
	On Institution Investments				2,144	4	0	
	On Deposit Account				232	15	2	
	Income Tax recovered for the year 1953–54				409	9	9	
2,807								2,786
9,720	Examination Fees for the April and October 1954 Examinations					..		10,236
213	Higher National Certificates—receipts less expenses					..		263
1,664	Library Fund Donations					..		1,913
—	Research—*see page 627*					..		622
94,579	TOTAL INCOME					..		97,326

Deduct EXPENDITURE.

1953–54 £		£	s.	d.	£	s.	d.	£
16,396	House and Establishment Charges				16,786	4	4	
	Repairs and Renewals Reserve—							
5,500	Amount provided for the year, *page 627*				4,000	0	0	
21,229	Salaries and Wages (including staff pension premiums)				20,041	9	2	
4,053	Stationery, Postage, etc.				5,081	1	7	
19,179	Publications—*see page 627*				17,457	14	2	
60	Research				—	—	—	
4,583	Library				4,676	7	5	
438	Library Reorganization				—	—	—	
7,527	Examination Expenses				8,686	6	0	
	Grants and Contributions—	£	s.	d.				
	Local Associations and Overseas Groups	5,075	10	7				
	Overseas Advisory Committees	85	9	0				
	Other Bodies	110	4	7				
6,004					5,271	4	2	
140	Annual Dinner				144	5	8	
1,053	Conversazione				1,731	0	6	
413	Legal and other Professional Charges				830	6	5	
210	Public Relations Committee Expenses				787	6	6	
1,603	Engineering Conferences				1,455	9	3	
1,352	Travelling Expenses to Committees				1,377	10	11	
662	Other Expenses				344	8	3	
450	Institution Building—Sinking Fund Allocation				450	0	0	
424	Coronation Expenses				—	—	—	
91,276	TOTAL EXPENDITURE							89,120
Surplus 3,303	Balance, being Surplus for the Year carried to Balance Sheet							£8,206

Note: Expenditure amounting to £2,000 (not included above) incurred on the Institut premises and on secretarial organization has been met out of a bequest of £3,000 received the late Mrs. E. M. Harvey.

OBITUARY

ROBERT GLOVER BAXTER, O.B.E., M.A., who died at his home in Westcliff on the 16th January, 1955, was born on the 31st May, 1901.

He was educated at Liverpool Institute, and at Downing College, Cambridge, of which he was a Foundation Scholar.

He received his engineering training under A. E. Jackson, Borough Engineer of Southport.

From 1925 he was Assistant Engineer at Salford. In 1930 he was appointed Chief Engineering Assistant at Hampstead. He subsequently became Deputy Borough Engineer at Luton, and later Assistant Borough Engineer at Brighton. In 1938 he was appointed Borough Engineer and Surveyor at Southend, a post he held until his death.

Mr Baxter was elected an Associate Member in 1927, and was transferred to the class of Members in 1946. He was also a Member of Council. He was a Fellow of the Royal Institution of Chartered Surveyors, a Member and Vice-President of the Institution of Municipal Engineers, and a Member of the Institute of Public Administration. He submitted various Papers to the Institution of Municipal Engineers.

He is survived by his wife and two children.

HUBERT CECIL BOOTH, who died in Croydon on the 14th January, 1955, was born on the 4th July, 1871.

He was educated at Gloucester County School, and received his engineering training at the Central Technical College, now known as the City & Guilds Engineering College, London.

He began his professional career in the drawing office of Maudslay, Sons and Field.

From 1894 to 1898 he was engaged in the design and construction of "Great Wheels" varying from 270 ft to 300 ft in diameter, for amusement parks in London, Blackpool, Paris, and Vienna.

In the following year he designed a structural steel factory in Belgium and managed it.

In 1900 he began a consulting practice in London. In 1901 he invented the vacuum cleaner, and the success of his discovery laid the foundation of the vacuum cleaner industry throughout the world. Later he became Chairman and Managing Director of the British Vacuum Cleaner and Engineering Co.

From 1903 to 1940 he was in general practice, handling the design and construction of steel railway bridges, factory plant, and other structural steel work. During World War I he was engaged on the installation of many vacuum-cleaning plants in high-explosive factories. One particular application of his invention was in the cleaning of the Crystal Palace at Sydenham for the Admiralty during a spotted fever epidemic. The epidemic subsided almost immediately.

From the end of the 1914–18 war he was concerned in the design of structural steelwork for factories and other buildings. He also designed many suspension bridges in Burma, India, and South Africa, and many over- and under-bridges both in steel and concrete for various railway companies in Britain.

He was elected an Associate Member in 1897, and was transferred to the class of Members in 1910. He was also a Fellow of the City and Guilds of London Institute.

ROBERT JAMES BOYD, M.E., who died at Lane Cove, New South Wales, on the 27th December, 1953, was born at Broughton Vale, New South Wales, on the 16th August, 1876.

He was educated at Sydney University and started his engineering career as draughtsman in the Sewage Construction Branch of the Public Works Department, Sydney, New South Wales.

From 1902 to 1907 he was assistant engineer for the New South Wales Government Railways, and from 1907 to 1913 engineer to Messrs Gummow Forest & Co. After two years in private practice he returned to the Department of Public Works, Sydney, and was engaged on design work for the construction of the Metropolitan Railway. In 1920 he was appointed Principal Design Engineer, Sydney Underground Electric Railway, a post which he held until 1934 when he became Inspecting Engineer for Steel Bridges, N.S.W. Railways.

In 1937 he accepted the position of Chief Designing Engineer, Department of Irrigation, Water Supply and Sewerage, Queensland. He retired from that post in 1941 and returned to Sydney to become a Consulting Engineer.

In that same year he was awarded the Peter Nicol Russell Memorial Medal.

He was elected an Associate Member of the Institution in 1907 and was transferred to the class of Members in 1922, he was also a Member of the Institution of Engineers, Australia, serving as President of that Institution in 1936.

He is survived by his sister Miss C. Margaret Boyd.

HERBERT CHATLEY, D.Sc.(Eng.), who died on the 17th January, 1955, was born on the 17th May, 1885.

He was educated at Cambridge House Grammar School, North London, and the Northern Polytechnic of London University. After a short period as a pupil engineer he obtained an appointment in the Borough Engineer's Department at Fulham. From 1906 until 1908 he was a lecturer in civil engineering at the Municipal College, Portsmouth. He then became

Professor of Civil Engineering at Tongshan Engineering College, North China, a post which he held until 1915.

From 1915 to 1916 Dr Chatley served with the Nanking-Hunan Railway as a District Engineer. In 1916 he joined the Whangpoo Conservancy Board, Shanghai, as Assistant Engineer, undertaking important dredging works and making valuable studies of the mud and silt in the Whangpoo and Yangtze estuaries. His work included supervising the construction of locally built dredging plant. He became Engineer-in-Chief in 1928, retiring from that position in 1937, when he returned to London to practise as a consultant.

On retiring from China Dr Chatley was awarded the Chinese Order of the Brilliant Jade in recognition of his services.

Dr Chatley was an authority on dredging and dredging plant; he was the Author of many publications on the subject. He also wrote on the theory of flight and on rocket propulsion.

In 1940 he joined the Department of the Civil Engineer to the Admiralty as Superintending Civil Engineer, in which capacity he was closely concerned with the Mulberry Harbour scheme. For his work while with the Admiralty the French Government made him an officer of the Legion of Honour. He left the Admiralty service in 1946 and resumed consulting practice.

Dr Chatley was elected an Associate Member of the Institution in 1920, and was transferred to the class of Members in 1928. He presented several Papers * to the Institution and was awarded a Telford Premium on four occasions.

He was also a member of the Institution of Civil Engineers of Ireland, an associate fellow of the Royal Aeronautical Society, and an associate of the Institute of Physics.

Dr Chatley is survived by his wife and three daughters.

LIEUT-COLONEL PERCY JOHN COWAN, M.B.E., who died on the 15th November, 1954, at the age of 78, was born at Surbiton, Surrey, on the 17th April, 1876.

He was educated first at Lausanne, then at Rugby School. For 3½ years he studied engineering at University College, London. After completing his practical training by serving for 3 years at the Great Northern Loco-

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* "Silt." Min. Proc. Instn Civ. Engrs, vol. 212 (1920–21), p. 400.
"The Constitution of Clay-Mud." S.E.P. No. 52 (1927).
"Problems in the Theory of River Engineering." S.E.P. No. 71 (1929).
"Energy Considerations in Dredging." S.E.P. No. 125 (1932).
"The Principles of Drag-Suction Dredging." J. Instn Civ. Engrs, vol. 12, p. 185 (June 1939).
"Hydrology of the Yangtze River." J. Instn Civ. Engrs, vol. 14, p. 227 (April 1940).
"Dredging Machinery." J. Instn Civ. Engrs, vol. 24, p. 72 (April 1945).

motive Department, Doncaster, he became Running Shed Foreman of this department and, later, District Locomotive Superintendent, Lincoln.

From 1900 to 1901 he was concerned with inspection in the U.S.A of locomotive contracts for the Great Northern Railway, Egyptian State Railways, Sudan Military Railways, and steel rolling stock and other railway material for Egyptian Railways. The following year he inspected rolling stock and other railway material for Egyptian State Railways.

In 1903 Colonel Cowan was awarded the George Stephenson Medal and Telford Premium for his Paper[1] on "American Locomotive Practice." In 1918 he was awarded the Willans Premium by the Institution of Mechanical Engineers for his Paper on "Feed Heating and Superheating of Locomotives."

After a period as Assistant Superintendent, Locomotive Department, Egyptian State Railways, from 1903 to 1905, he joined the editorial staff of *Engineering*, on which he served until the outbreak of the 1914–18 war, during which he saw service, first, in Mesopotamia in charge of the installation of refrigeration, electricity supply, and pumping plant and, later, in Kut where he was responsible for reorganizing and increasing output of the locomotive department and all mechanical plant to the Persian frontier.

In the Afghan war of 1919 he was in charge of the Electrical and Mechanical Workshops of the North-Western Frontier Force.

"Mentioned in Dispatches" in 1917, he was the same year awarded the M.B.E.

Colonel Cowan was elected an Associate Member in 1903 and transferred to the class of Members in 1927.

After the cessation of World War I, he returned to the editorial staff of *Engineering*, on which paper he served as editor from 1924 until his retirement in 1938.

During the latter part of World War II, Colonel Cowan served as County Army Welfare Officer for Surrey.

SIR MAURICE EDWARD DENNY, BT, K.B.E., LL.D., D.L., S.B., J.P., who was born at Dumbarton in February 1886, died on the 2nd February, 1955.

He was educated at Tonbridge School and after three years in Switzerland and Germany he went to Massachusetts Institute of Technology, where, after 4 years he graduated as Bachelor of Science. After an apprenticeship to Doxford and Sons, Sunderland, he gained further practical experience in the family shipyard and in 1911 became a partner in the firm. When the firm became a limited company in 1917 he was appointed a director and succeeded to the Chairmanship in 1922.

During the 1914–18 war he served in the Machine Gun Corps and was created a Commander of the Order of the British Empire in 1918.

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[1] Min. Proc. Instn Civ. Engrs, vol. 154 (1902–03), p. 38.

Apart from his main interest, which was in Shipbuilding and Marine Engineering, Sir Maurice served on the boards of the Union Bank of Scotland, and Blackburn & General Aircraft Ltd as well as a number of companies engaged in industries allied to that of his own firm. In 1935 he was elected President of the Institute of Marine Engineers. He also served as Vice-President of the Institution of Naval Architects, to whom he had previously presented several technical Papers, and of the Junior Institution of Engineers. In 1940 he was elected President of the Shipbuilding Conference.

During the 1939–45 war Sir Maurice's firm built more than thirty vessels for the Royal Navy and converted two merchantmen to aircraft carriers.

He was a keen ornithologist and numbered model-making among his other hobbies.

He was elected an Associate Member of the Institution in April 1918 and was transferred to the class of Members in April 1924. He succeeded to the baronetcy in 1936. In 1952 he retired, relinquishing the Chairmanship of his company to a younger cousin whilst he himself became President.

JOHN TERRACE, who died at Sydenham, London, on the 20th January, 1955, was born on the 20th September, 1874.

He was educated at High School, Arbroath, and at F. C. Normal School, Glasgow, and he received his practical training with Ashmore, Benson, Pease and Company.

From 1897 to 1902 he was Constructional Engineer with Humphreys and Glasgow, and was connected with the erection of carburetted water-gas plants.

For the next 6 years he served as Assistant Engineer to the Tottenham and Edmonton Gas Light and Coke Co., during a complete reconstruction and extension of their works.

In 1908 he was appointed Engineer and Manager to the Great Grimsby Gas Co., and was responsible for the construction of a new retort house and a new fitting shop.

From 1919 until his retirement in 1931 he was Chief Engineer of the South Suburban Gas Co., and was responsible for the reconstruction of their Lower Sydenham works.

Mr Terrace was elected a Member in 1924. He was also a Past President and Honorary Secretary of the Institution of Gas Engineers, a Member of the Institution of Chemical Engineers, and a Fellow of the Institute of Fuel.

He is survived by a son and a daughter.

CORRIGENDA

Proceedings, Part I, May 1955.

Paper No. 6034 (Young and Landau)

p. 303. *For* where $= k_b\sigma_e$

$$\sigma_k\sigma_e = \frac{\pi^2 E}{12(1-\nu^2)} \cdot \left(\frac{t}{b}\right)^2$$

read $\sigma_k = k_b\sigma_e$

$$\text{where } \sigma_e = \frac{\pi^2 E}{12(1-\nu^2)} \cdot \left(\frac{t}{b}\right)^2$$

p. 304. Fig. 2. Ordinates should be k_b not k

p. 308. Table 2, column 1, line 2. *For* " max min " *read* " max/min ratio "

p. 309, Fig. 4. " Note " *should read* " Note : for $a/b = 0{\cdot}75$ to 1·50, k depends mainly on ρ."

In caption, for " Inability " *read* " Instability "

p. 310, para. 2, line 4. ***Vor*** v ***read*** ν

p. 318, Fig. 10c. *For* " VALUES OF $h/1$ " *read* " VALUES OF h/t "

p. 320, Fig. 11. *For* " v " *read* " ν " *in two places*

p. 322, Table 3. *In column* (7). For 5′ 0″ web " 2 Ls $3\frac{1}{2}''\times2\frac{1}{2}''\times\frac{3}{8}''$ " *refers to* 2′ 6″ stiffener spacing ; " Nominal " *to* 5′ 0″ *and* 7′ 6″ stiffener spacing. For 7′ 6″ web, " 2 Ls $5\frac{1}{2}''\times3''\times\frac{3}{8}''$ " *refers to* 2′ 6″ spacing ; " 2 Ls $5\frac{1}{2}''\times3''\times\frac{3}{8}''$ " *to* 5′ 0″ spacing ; *and* " Nominal " *to* 7′ 6″ spacing.

p. 324, Equation (1). *For* " $-\,6{\cdot}5\alpha^2$ " *read* " $+\,6{\cdot}5\alpha^2$ "

p. 325, Fig. 13. *For* " DUMAS " *read* " DUBAS "

p. 326, Table 4. *Add below Table :* The factor 2 in the second column applies only for $s<h/5$. This factor is independent of and additional to the safety factor of 2.

p. 327, line 2. ***For reference*** **16** ***read*** **17**

$$\left.\begin{array}{l}\text{p. 331, para. 2, second equation}\\ \text{p. 332, para. 2, second equation}\\ \text{p. 332, para. 7, first equation}\end{array}\right\}\ \textit{For}\ \text{“ }v\text{ ” }\textit{read}\ \text{“ }\nu\text{ ”}$$

p. 333, para. 2, lines 4, 5, 6. *For references* 17, 18, 19 *read* 18, 19, 20.

ANNUAL GENERAL MEETING

7 June, 1955

DAVID MOWAT WATSON, B.Sc., President, in the Chair

The President, having moved, and the Meeting having agreed, that the Report of the Council which, as intimated in the May number of *Chartered Civil Engineer*, had been available to Members upon request to the Secretary, be taken as read, said that the Report revealed that it had been a very active year for the Institution. There had been no less than thirty-five meetings, including three Joint Meetings, at which Papers were presented.

All awards for Papers would be announced at a meeting early next Session. Assessment of the relative value of Papers was slow and required longer nowadays when the number of Papers was greater.

Several Conferences had been held during the year, including the Conference on Engineering Education at Zurich, the Conference of Engineering Institutions of the British Commonwealth, the Conference on Civil Engineering Problems in the Colonies, and the Conference on Highway Problems in the Colonies. All those Conferences had been well attended.

During the year he had had the pleasure of attending the annual dinners of all ten of the Local Associations, and he was glad to have had the opportunity of meeting the Chairmen, Officers, and Members of the Local Associations, to whose work and interest in Institution affairs they all owed so much. It was one of the satisfactory marks of the liveliness of the Institution that not only the Local Associations at home but the Overseas Associations and Joint Overseas Groups were so active.

The Council welcomed the participation of the Institution of Mechanical Engineers in the Joint Part I Examination, and they hoped that other engineering societies would also join. Indeed, he had heard only recently from the Secretary that the Institution of Municipal Engineers intended to participate.

The Accounts were set out in detail in the Report. The President pointed out that there was an excess of revenue over expenditure of £8,206, which was due mainly to increases in revenue from advertising and many small economies in printing. It would be a mistake, however, if it were so be read as an indication that the Institution had plenty of money. There were so many activities in the world of civil engineering which the position of the Institution required and prestige demanded that the tendency was to have to do more and more and to take a greater part in uch work. The Institution's activities were no longer confined to the United Kingdom or even to the Commonwealth, as had once been the case. All these growing activities helped the Members, sometimes directly, ometimes indirectly, but all cost money, and the increased costs would have to be faced.

It was gratifying that the Rockefeller Foundation had allocated further funds for administration by the Institution to provide Bursaries in Public Health Engineering. The original scheme had been for three years, expiring in 1955, but the new arrangement enabled it to continue until 1960.

A Conversazione for Graduates and Students had been held last October and one was to be held again next October. This function had been a great success and was, of course, additional to the main Conversazione usually held in June.

The formation of the British Nuclear Engineering Conference was a matter of great interest. The Conference would be a permanent arrangement, and had been formed by the co-operation of the Institution and the Institutions of Mechanical, Electrical, and Chemical Engineers and the Institute of Physics. Its headquarters and secretariat would be in the Institution and Mr Alexander McDonald was the Secretary of the Conference. The five Institutions would arrange for the presentation of Papers, and the meetings and discussions would be open to all Members of the Institutions. A Journal would be published and the first issue, in January, 1956, would contain a symposium of lectures to be delivered at the Institution on 30 November.

The new Secretary had almost completed his first year in office and was rapidly mastering the intricacies of the Institution's activities, which were not inconsiderable.

A portrait of Mr Graham Clark had been painted posthumously by John Whitlock and would be hung in the Reading Room.

Mr W. T. Shaddock observed that there was a reference in the Report to the effect that a Joint Committee had been set up, consisting of two representatives from each of the Institutions of Civil, Mechanical, and Electrical Engineers and the Engineers' Guild. Did that imply that the Council of the Institution recognized the Engineers' Guild to be *the* appropriate, and not merely *an* appropriate, technical association for members of the Institution ?

The President replied that the Institution did not look upon the Engineers' Guild as being a technical institution. There were matters that were very closely allied, perhaps, to the functions of both, and this Committee existed for the purpose of discussing them. But the Institution were bound by their Charter and they were meticulously careful not to go beyond the bounds of the Charter. The Guild had its own function, which was entirely different from the function of the Institution.

There being no further questions, the motion for the reception and adoption of the Report was agreed to.

The President announced the following report of the Scrutineers on the ballot for the election of the Council for Session 1955–56.

THE INSTITUTION OF CIVIL ENGINEERS
COUNCIL 1955–1956

President

WILLIAM KELLY WALLACE, C.B.E.

Vice-Presidents

Harold John Frederick Gourley, M.Eng.
Sir (Frederick) Arthur Whitaker, K.C.B., M.Eng.
Professor Alfred John Sutton Pippard, M.B.E., D.Sc., F.R.S.
Arthur Clifford Hartley, C.B.E., B.Sc.

Other Members of Council

*Maurice Edward Adams, O.B.E.
Professor John Fleetwood Baker, O.B.E., M.A., Sc.D., D.Sc.
Arthur Borlase, T.D. (*South Wales & Monmouthshire*).
*Professor William Fisher Cassie, Ph.D., M.S. (*Northern Counties*).
Walter Edmund Doran, O.B.E., B.A., B.A.I.
Johan Hendrik Durr, B.Sc. (*N. & S. Rhodesia*).
Arthur Floyd, C.B.E., B.Sc.
Ralph Freeman, C.B.E., M.A.
Sir George (Herbert) Fretwell, K.B.E., C.B.
Angus Anderson Fulton, B.Sc., F.R.S.E.
*John Edward Harben, M.Eng. (*North Western*).
Harold John Boyer Harding, B.Sc.
*Kenneth Palmer Humpidge, B.Sc. (*Colonies*).
Nathaniel Harvey Hunt (*India & Pakistan*).
Sir Claude (Cavendish) Inglis, C.I.E., M.A.I., F.R.S.
John Holmes Jellett, O.B.E., M.A.
*Robert Ferguson Legget, M.Eng. (*Canada*).
William Linn (*Glasgow & West of Scotland*).
Robert McCreary, O.B.E., M.C., B.A., B.Sc. (*Northern Ireland*).
George Matthew McNaughton, C.B., B.Sc.
Sir Herbert (John Baptista) Manzoni, C.B.E.
Donald Stuart Gore Marchbanks, D.S.O., M.B.E. (*New Zealand*).
Reginald William Mountain, B.Sc.
*John Lawrence Paisley, M.B.E., M.Eng.
Thomas Angus Lyall Paton, B.Sc.
Adrian Benson Porter.
Joseph Rawlinson, C.B.E., M.Eng.
Charles Arthur Risbridger, B.Sc. (*Midlands*).
Brendan Francis Saurin, B.Sc.
Henry Schrader, M.Sc., B.Sc. (*South Africa*)
George Edward Scott, O.B.E., V.D., M.Eng., B.Sc. (*South Western*).
James Herbert Siddons, M.Eng. (*Ceylon*).
Percy Edward Sleight, M.Eng. (*Southern*).
Hubert Shirley Smith, O.B.E., B.Sc.
Marcus George Russell Smith, M.B.E., B.Sc.
Maj.-Gen. Sir Clive Selwyn Steele, K.B.E., D.S.O., M.C., V.D., B.C.E. (*Australia*).
*James Gilbert Taylor (*Yorkshire*).
Charles Bruce Townend, C.B.E., B.Sc.
John Harold Wallace Turner, B.Sc.
Sir Hubert (Edmund) Walker, C.B.E. (*Colonies*).
*Godfrey Maurice Wheat (*Colonies*).
John Letham White, C.B.E., D.L., B.Sc. (*Edinburgh & East of Scotland*).
George Ambler Wilson, M.Eng.

* Newly elected.

Mr W. R. Gilcriest moved, **Mr A. A. Osborne** seconded, and the meeting agreed that thanks be accorded to the Scrutineers and that the ballot papers be destroyed.

Mr D. A. Brown acknowledged the resolution on behalf of the Scrutineers. As a matter of interest, 9,040 papers had been sent out and 1,886 returned ; the figures for the previous year had been 9,523 and 1,867. Considered as a percentage of the Corporate Membership, those figures represented 73 per cent and 14 per cent. There had been only 46 invalid papers, as against 69 the previous year.

He wished to record the Scrutineers' appreciation of the assistance given by members of the permanent staff of the Institution.

Mr A. N. M. Robertson moved, **Mr W. Fairley** seconded, and the meeting carried, a resolution that Mr J. A. Johnston, Member, be re-appointed Honorary Auditor for the current financial year ; and that Sir Alan Rae Smith be re-appointed the professional Auditor for the current financial year.

Mr R. J. Ashby moved, **Miss Letitia Chitty** seconded, and the meeting carried with acclamation, a vote of thanks to the President for his conduct of the business as Chairman of the meeting.

The President acknowledged the vote of thanks and the meeting then terminated.

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